SR 167, C.S. 1765/6, OL-2305
15th Avenue SW to 15th Avenue NW
HOV Lanes
Geotechnical Report

Prepared for

Washington State Department of Transportation
Materials Branch

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Chapter 1 -- Introduction

This report summarizes results of geotechnical explorations and foundation design studies completed for the SR-167, CS1765/6, OL-2305, 15th Avenue SW to 15th Avenue NW High Occupancy Vehicle (HOV) lane widening project. The geotechnical program described in this report was conducted for the Washington State Department of Transportation (WSDOT), as part of On-Call Geotechnical Services Agreement Y-6050. Geotechnical work for this HOV widening project was authorized by WSDOT on November 14, 1997 as Task Assignment No. AD.

Purpose and Scope

The purpose of the explorations and foundation design studies performed under this task order was to provide information that would assist WSDOT in the selection of foundation types, sizes, and embedment depths for the widening of three bridge structures within the project limits. The bridges are:

- Bridge No. 167/112 N-E Ramp
- Bridge No 167/112 E Ramp
- Bridge No. 167/112 W-N Ramp

The scope of work for this task order included:

- Reviewing existing information for the project, including the geology of the area, original design drawings, and subsurface information from previous WSDOT explorations in the area
- Recording blowcounts from Standard Penetration Tests (SPTs) and visually classifying soil samples recovered during drilling of 11 test holes at proposed bridge foundation locations
- Conducting laboratory tests on a limited number of soil samples recovered during the field explorations to aid in the classification of soil types
- Characterizing subsurface soil and groundwater conditions based on the results of the field explorations and laboratory tests
- Reviewing the regional geology and seismicity for the project site, including ground motions that are appropriate for soil response evaluations and bridge design
- Establishing foundation design considerations and requirements for each bridge, consisting of

⇒ subsurface conditions, relevant soil properties, and liquefaction susceptibility

⇒ the axial capacity of driven piles and drilled shafts

⇒ settlement of driven piles and drilled shafts under anticipated service loads

⇒ soil parameters for use in the lateral analyses of driven piles and drilled shafts

⇒ the effects of seismic loading on the capacity of driven piles and drilled shafts

⇒ the stability of abutment fills during seismic loading, and

⇒ allowable bearing pressures for abutment footings

- Preparing this summary of results from the geotechnical explorations and foundation design studies

Results of these geotechnical explorations and foundation design studies are presented in six chapters following this introductory chapter. The first two chapters, Technical Data and Area Geology and Seismicity, pertain to the entire project. The next three chapters (N-E Ramp Structure, E-Ramp Structure, and W-N Ramp Structure) provide information specific to each of the three bridge structures. The final chapter presents references for the report. Figures, soil test hole logs, and laboratory data for each bridge are presented at the end of chapter addressing the bridge.

Background

The general project involves widening of SR-167 to handle HOV lanes between 15th Avenue SW and 15th Avenue NW (Milepost 13.73 to 15.76). This widening project is located in King County, Section 23, Township 21 North, Range 4E West Meridian near the City of Auburn, Washington. Figure 1-1 shows a vicinity map for the project.

Area Description

The bridges are located on the western side of the Kent Valley approximately 2 km (1.2 miles) west of the City of Auburn and 32 km (20 miles) south of Seattle. This area is relatively flat with the primary relief being the 8- to 9-m (26 to 30 ft) approach fills required by SR-167 to pass over SR-18. Natural ground surface elevations range from 18 to 21 m (60 to 70 ft) National Geodetic Vertical Datum (1978).

Approximately 1 km (0.6 miles) to the west of the project site, the ground surface climbs to an uplands area. The elevation of the uplands area is roughly 100 m (330 ft) above the valley. SR-5 is located approximately 4 km (2.5 miles) west of the project site.

Bridge Descriptions

Three bridge structures (N-E Ramp, E Ramp, and W-N Ramp) will be widened as part of this HOV project. The N-E Ramp and W-N Ramp provide access from 15th Avenue SW and eastbound SR-18, respectively, to northbound SR-167. The E Ramp is the mainline bridge for northbound traffic on SR-167.

The three bridges were constructed in the 1970's. The size and foundations systems for the existing bridges are as follows:

- **N-E Ramp:** This bridge is located on an access ramp from 15th Avenue SW approximately 100 m (330 ft) south of the W-N Ramp. It is approximately 83 m (273 ft) in length and 8 m (26 ft) in width. The bridge is supported on two interior piers with each pier consisting of a single column. The columns are supported on a pile foundation system consisting of a pile cap and creosote-treated timber piles with as many as 28 piles at each cap. The timber piles have a capacity of 360 kN (40 tons). End abutments for the bridge are supported by a 1.5-m (5 ft) wide strip footing located approximately 3 m (10 ft) below the roadway surface. The allowable bearing pressure on the footing is 290 kPa (3 tsf).

- **E Ramp:** This bridge is approximately 105 m (343 ft) in length and 12 m (40 ft) in width. It is supported by four interior piers with each pier consisting of two columns. Each column is located on a pile cap. A 6-pile group supports the pile cap. Piles are 490 kN (55 ton) concrete piles driven approximately 12 to 15 m (40 to 50 ft) below the ground surface. End abutments for the bridge are supported by a 1.5-m (5 ft) wide strip footing located approximately 3 m (10 ft) below the roadway surface. The allowable bearing pressure on the footing is also 290 kPa (3 tsf).

- **W-N Ramp:** This bridge is located approximately parallel to the E-Ramp, with the west side of the W-N Ramp from 3 to 9 m (10 to 30 ft) from the east side of the E-Ramp. The length of this bridge is approximately 101 m (331 ft), and its width varies from approximately 14 m (46 ft) at the south end to 12 m (40 ft) at the north end. This bridge is also supported by four interior piers with each pier consisting of two columns. As with the E-Ramp, each column is located on a pile cap, which in turn is supported by a 6-pile group. Piles are 490 kN (55 ton) concrete piles driven to approximately the same depths as piles driven at the East Ramp. End abutments for the bridge are also supported by a 1.5-m (5 ft) wide strip footing located approximately 3 m (10 ft) below the roadway surface and having an allowable bearing pressure of 290 kPa (3 tsf).

Existing Geotechnical Information

Geotechnical explorations were completed between 1969 and 1971 by WSDOT for the original design of the three structures. The exploration program resulted in 10 test holes advanced to depths of as much as 44 m (144 ft) below the existing ground surface, which at the time was located at approximate elevation 18 to 21 m (60 to 70 ft). SPT blowcounts were recorded in each test hole. A limited amount of laboratory test information was also obtained as part of this original program.

In 1991 a second exploration program was completed for WSDOT in the same area (Terra, 1991). This program included a test hole drilled at each of the abutments for the W-N Ramp and a single test hole between the south abutments of the W-N Ramp and the E Ramp. Hollow-stem auger methods were used to advance the test holes to a maximum depths of approximately 37 m (120 ft) below the top of the approach fill. The approach fill is located approximately at elevation 27 m (90 ft). SPTs were obtained during the drilling program; laboratory classification tests were conducted on representative soil samples recovered from the test holes.

Proposed Bridge Modifications

WSDOT plans to widen each of the bridge structures to accommodate an additional traffic lane. The width of the widening will be 3.7 m (12 ft) for the N-E Ramp, 5.5 m (18 ft) for the East Ramp, and from approximately 1.5 to 3.7 m (5 to 12 ft) for the W-N Ramp. Single columns will be added along the existing pier lines to support the widened section of the bridges. The new columns will be supported on either driven piles or drilled shafts, depending on the results of this geotechnical exploration and foundation design study.

Preliminary column loads were provided by WSDOT for the bridges. These loads, which are summarized in Table 1-1, were used for preliminary bridge foundation settlement analyses. Live loads were also provided by WSDOT's bridge design group. However, during a progress review meeting with WSDOT's geotechnical engineers, it was decided that the foundation settlement analyses would be conducted only for the service loads.

Table 1-1. Preliminary Bridge Column Loads

Bridge	Pier No.	Load
E-Ramp	2	1,749 kN (395 kips)
	3	2,117 kN (478 kips)
	4	1,807 kN (408 kips)
	5	1,408 kN (318 kips)
W-N Ramp	2	1,151 kN (260 kips)
	3	1,351 kN (305 kips)
	4	961 kN (217 kips)
	5	589 kN (133 kips)
N-E Ramp	2	3,228 kN (729 kips)
	3	3,228 kN (729 kips)

Approach fills will be widened to accommodate the extra bridge lane. The side slope for the widened embankment will be the same as the existing slope. Small wing walls may be used on some of the bridges to retain the added fill at the abutment end slope. WSDOT's geotechnical engineers will provide design requirements for the wings walls, if they are needed.

Project Team

This project was completed by a team of engineers from WSDOT, CH2M HILL, and subconsultant firms. The WSDOT project manager for this work was James G. Cuthbertson, P.E. The subconsultant firms included CivilTech Corporation, Terra Associates, Inc., and Soil Technology. Terra Associates and CivilTech were responsible for field inspection services during drilling and sampling of each test hole, as well as foundation design studies for the E-Ramp and W-N Ramp structures, respectively. Terra Associates also prepared a summary of geological conditions for the site. Soil Technology provided laboratory testing services. CH2M HILL was responsible for the foundation design studies at the N-E Ramp bridge, seismic studies, report preparation, and overall coordination of the project.

Information presented in this report has been reviewed by senior geotechnical engineers from CivilTech, Terra Associates, and CH2M HILL. An overall review of all components of the report was performed by WSDOT's project manager, Jim Cuthbertson, and chief foundation engineer, Robert Kimmerling.

Limitations

This report has been prepared exclusively for WSDOT and its contractors and consultants for the specific application to the proposed improvements to the N-E Ramp, E-Ramp, and the W-N Ramp bridge structures. Field work, laboratory testing, and geotechnical evaluations have been conducted for this task order in general accordance with locally accepted geotechnical engineering practice. No other warranty, express or implied, is made.

The recommendations contained in this report are based on existing test hole information and drawings provided by WSDOT, as well as the results of supplemental test holes drilled as part of this task order. Both the previous and new test hole logs indicate subsurface conditions only at the specific locations and at the time of sampling. They do not necessarily reflect strata variations that may exist between these locations nor do they necessarily reflect changes in groundwater conditions with time. If subsurface conditions different from those described in this report are noted during construction, CH2M HILL should be notified of the differences so it can review the information and determine if reevaluation of geotechnical recommendations given in this report is necessary.

The recommendations in this report are based on the proposed widening, as discussed in the following chapters for each bridge structure. If the nature or location of widening changes, the recommendations contained in this report should not be considered valid unless CH2M HILL reviews the changes and verifies or modifies the recommendations in writing. CH2M HILL is not responsible for any claims, damages, or liability associated with the subsurface data presented in this report or reuse of engineering analyses without the express written authorization.

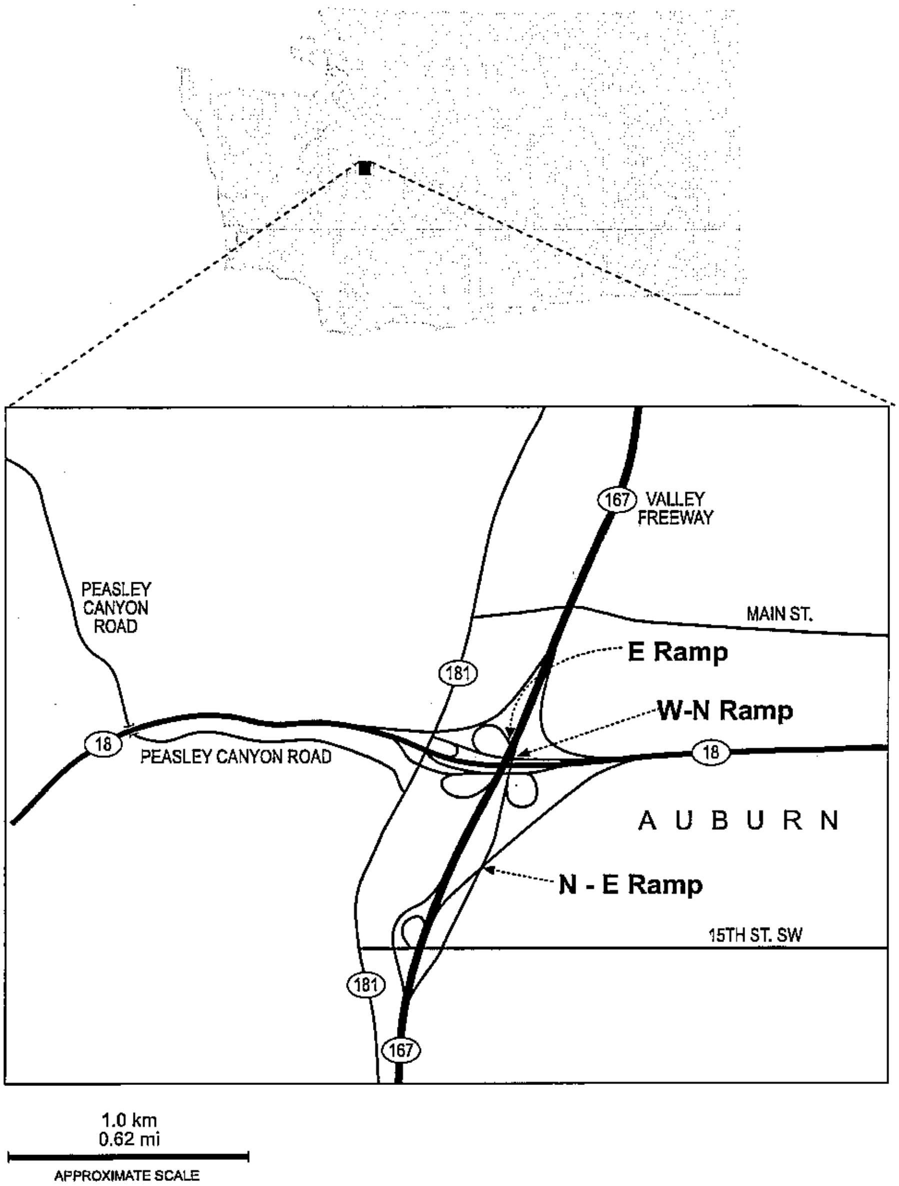

Figure 1-1. **Project Vicinity Map**

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SR-167, OL-2305
15th Ave. SW to 15th Ave NW
HOV Widening Project

Chapter 2 --Technical Data

This section of the report summarizes technical data upon which the foundation design studies are based. The information includes the results of field explorations and laboratory testing programs performed as part of this task order, as well as information collected during field explorations conducted between 1969 and 1971 and in 1991. No records were found for the original construction of the bridge foundations.

Field Explorations

The field explorations involved drilling and sampling of test holes, some of which extended more than 40 m (131 ft) below the ground surface. These test holes encountered soils that ranged from silts to gravels. Most of the soil profile consisted of layers of silty sands and gravelly sands.

1997 Drilling and Sampling

The 1997 drilling and sampling program was performed in November and December of 1997. Eleven test holes were drilled during the program. Drilling services were provided by WSDOT using its own crews and equipment. The maximum depth of drilling was 37 m (121 ft); the minimum depth was 12 m (40 ft). Five of the test holes were drilled from the highway shoulder on the top of the approach fill for the bridges, where the ground elevation was approximately 27 m (90 ft). The remaining six test holes were drilled along the shoulder of SR-18 at approximate elevation 18 to 21 m (60 to 70 ft).

The locations of test holes relative to the bridge plan drawings are given in Chapters 4, 5, and 6 with each bridge discussion. The designation "xx-97" indicates test holes completed as part of this task order. These locations were selected by WSDOT and CH2M HILL. WSDOT staked the locations prior to the start of the drilling program. Horizontal locations were established in the field by measuring to fixed components of the bridge (e.g., column location or bridge seat) using a hand tape. The ground surface elevations at the test hole locations were estimated from the preliminary plan drawings for the three bridges. Table 2-1 summarizes the approximate depths, locations, and elevations of the test holes.

Table 2-1. 1997 Test Hole Information

Bridge	Test Hole	Depth (m)	Station (m)	Offset (m)	Ground Surface Elevation (m)
N-E Ramp	H-1-97 H-2-97 H-3-97	12.7 36.7 12.3	6+53 7+21 7+47	8.5 R of CL 7.0 R of CL 6.4 R of CL	28 21 28

E-Ramp	H-4-97 H-5-97 H-6-97 H-7-97	36.7 15.2 24.8 12.3	158+72 159+25 159+40 159+57	9.5 R of CL 11.6 R of CL 8.5 R of CL 8.8 R of CL	23 21 23 30
W-N Ramp	H-8-97 H-9-97 H-10-97 H-11-97	14.0 35.2 24.5 12.7	9+35 9+48 10+03 10+38	11.0 R of CL 9.8 R of CL 9.1 R of CL 9.5 R of CL	28 23 21 29

Note: CL refers to centerline

Two types of drilling rigs were used to advance the test holes during the 1997 HOV widening project. The four test holes at the abutments on the N-E Ramp and the W-N Ramp bridges (H-1, H-3, H-8, and H-11) were drilled using WSDOT's CME 45C skid rig. The skid rig was used because of limited room at the required test hole location. A rubber-tire Longyear BK-80 was used to drill all the remaining holes.

Drilling was accomplished using either hollow-stem auger or mud rotary methods. A 100-mm (4 in), inner diameter hollow-stem auger was used for the shallow test holes. The annulus within the hollow-stem auger was filled with water in an effort to minimize the potential for heave. The mud rotary procedure used a 100-mm (4 in) or 76-mm (3 in) diameter casing advancer. Holes were typically started with the 100-mm casing and changed to 76-mm casing at depths of approximately 10 m (33 ft). Generally, drilling was accomplished without incident, although progress was difficult in the gravelly soils with the mud rotary method and heave became a problem with the hollow-stem auger system.

Standard penetration tests (SPTs) were conducted at approximately 1.5-m (5 foot) intervals in general accordance with ASTM D1586. The SPTs in all but one of the test holes were completed using WSDOT's automatic hammer. This hammer is equipped with a 0.62 kN (140 lb) weight and uses a free fall of 760 mm (30 in). Results of a recent energy monitoring program (ASCE, 1995) determined that this hammer has an efficiency of approximately 80 percent. Some of the SPTs in the one other hole were completed using a 0.62 kN (140 lb) safety hammer in combination with a rope-cathead lift procedure. The safety hammer was used when a hydraulic leak developed in the automatic hammer. Sample liners were not used in any of the SPTs.

Samples recovered during the SPTs were visually classified in the field by professional engineers from either Terra Associates or CivilTech, using ASTM D-2488. Following the classification of each sample, portions of the samples were sealed in plastic bags for storage and subsequent laboratory testing. Terra Associates and CivilTech prepared field test hole logs summarizing their visual classifications, SPT blowcounts, and other field observations for each test hole. Copies of these test hole logs are included at the end of Chapters 4, 5, and 6 for each bridge discussion.

Piezometers were installed in test holes H-2, H-4, and H-8 to allow for long-term monitoring of groundwater levels. The piezometers consisted of 25-mm (1 in) diameter solid PVC pipe with the bottom 1.5 m (5 ft) slotted. A 4-m (13 ft) long sand pack was placed around the slotted section of pipe. The top of the piezometer was completed with a locking steel cap.

Previous Exploration Programs

Two exploration programs had been conducted previously at the project site. The first program occurred between 1969 and 1971 for the design of the original bridge structures; the second occurred in 1991 as part of the preliminary phase of this widening project.

The original WSDOT program involved four test holes at the N-E Ramp, three test holes at the E Ramp, and three test holes at the W-N Ramp. These test holes were drilled and logged by WSDOT personnel. According to the test hole logs, the test holes were advanced by a variety of methods, including "chop and drive", rotary wash, and hollow-stem auger. It is understood from discussions with WSDOT engineers that SPTs were most likely conducted using the rope-cathead method with what WSDOT refers to as a slug weight. It is assumed that this weight was either a safety hammer or donut hammer. The following table provides a summary of the depths, locations, and elevations of the test holes drilled during WSDOT's original programs.

Table 2-2. Previous Soil Test Hole Information

Bridge	Test Hole	Depth (m)	Station (m)	Offset (m)	Elevation (m)
N-E Ramp	H-1-70 H-2-70 H-3-70 H-4-70	29.9 31.9 22.1 21.3	7+42 6+58 6+82 7+18	1.5 L of CL 1 R of CL 1.5 L of A Line 0.3 L of A Line	19.5 19.2 19.4 19.1
E Ramp	H-2-69 A-2-71 A-4-71 B-9-91 B-10-91	35.7 43.8 36.3 36.6 35.1	159+53 159+03 185+55 158+52 159+60	1 L of CL 18.3 R of CL 12.2 R of CL 7.6 R of LM 7 R of LM	19.5 20.9 19.5 ±29 ±29
W-N Ramp	H-3-69 A-3-71 A-5-71	37.0 37.7 34.9	10+23 9+78 9+35	-- 7.9 R of CL 8.8 R of CL	19.8 20.9 19.5

The more recent drilling program was completed by TAI in 1991. This program included test holes in the approach fills at both ends of the E-Ramp bridge and a single test hole approximately midway between the edges of the W-N Ramp and E Ramp structures on the south abutment. The test holes were drilled by Drilling Unlimited of Seattle, Washington using hollow-stem auger methods. Test holes were drilled to depths of 30 to 36.5 m (100 to 120 ft) below the abutment elevation (approximate elevation 29 m; 96 ft). SPTs were performed every 1.5 m (5 ft) using rope-cathead procedures with a safety (?) hammer. The locations of these test holes are also given in Table 2-2.

Copies of test hole logs from these previous programs are included at the end of Chapters 4, 5, and 6 with each bridge discussion.

Laboratory Testing

Limited numbers of laboratory tests were conducted to assist in classifying soils recovered during each of the drilling and sampling programs. Inasmuch as soil within the profile is primarily cohesionless, the laboratory testing programs generally focused on grain-size analyses and water content determinations.

1997 Laboratory Testing Program

The 1997 laboratory testing program was conducted by Soil Technology of Brainbridge Island, Washington. Soil Technology completed 10 mechanical grain-size analyses (ASTM D-422), 14 No. 200 wash sieve analyses (ASTM D-1140), and two Atterberg Limit tests (ASTM D-4318). Samples were selected for testing by CivilTech, Terra Associates, and CH2M HILL based on each organization's review of the soil profile at the bridge for which it had responsibility. CH2M HILL also reviewed the CivilTech and Terra Associate assignments for overall consistency and to confirm that information would be available to assist CH2M HILL in its liquefaction analyses.

Table 2-3 summarizes the numbers and types of tests completed for each bridge. Results of these laboratory tests are included at the end of Chapters 4, 5, and 6 with each bridge discussion.

Table 2-3. 1997 Laboratory Testing Program

Bridge	Mechanical Grain-Size Analyses	No.200 Wash Sieve	Atterberg Limits
N-E Ramp	3	6	-
E Ramp	4	3	1
W-N Ramp	3	5	1

Previous Laboratory Testing

Limited numbers of laboratory tests were also completed for the 1969-1971 WSDOT program and the 1991 Terra program. These programs focused on soil classification testing. WSDOT also performed six consolidation tests on silty soils and 18 triaxial compression tests during its program. Terra conducted two consolidation tests on silty soil samples.

Results of the WSDOT and Terra laboratory test programs are included at the end of Chapters 4, 5, and 6 with each bridge discussion.

Chapter 3 -- Regional Geology and Seismicity

A review of the regional geology and seismicity for the project area was performed. The intent of this review was twofold: (1) to identify mechanisms that resulted in or influenced the makeup or consistency of soil within the depth range that bridge structure foundations would likely be installed, particularly with respect to any unique conditions that could affect either the construction or performance of the bridge foundations, and (2) to establish the level of seismic loading to use for seismic design. WSDOT currently uses a 10 percent probability of exceedance in 50 years as its design basis.

Regional Geology

The project site is located in the Green-Duwamish River Valley in the central Puget Sound Lowland physiographic province. The geology of the Puget Sound Lowland is the product of several complex geologic processes extending over a long period of time. The area geology as described in the Geological Survey Professional Paper 672 - *Geology of the Renton, Auburn, and Black Diamond Quadrangles, King County, Washington* by Mullineaux (1970) is summarized below.

Glacial Sedimentation

Pleistocene deposits in the Puget Sound lowland record at least four glaciations separated by interglacial intervals. The three pre-Vashon glacial and the two intervening interglacial episodes are named for formations that crop out in the Puyallup Valley.

The oldest deposits recognized as Pleistocene are named Orting Drift and usually consist of stony till and outwash that lie on Tertiary formations along the Green River. The Orting Drift consists predominantly of brown sand and gravel that contains lenses and sheets of till, silt, and clay. Typically, the Orting till is very compact and stony. The Orting drift deposits are more than 60 m (200 ft) thick.

The intermediate drift formation is next younger than the Orting deposit. This stratigraphic unit, which consists mostly of two fine-grained, clay-rich till members separated by 15 to 55 m (50 to 180 ft) of stratified sediments, crops out along the Green and White River valley walls. The intermediate drift lies between Orting Drift and Puyallup Formation. Its position thus suggests equivalence with drift of the Stuck Glaciation, but it cannot be traced to that formation and is strikingly dissimilar to the typical Stuck Drift. The intermediate drift consists of fine-grained till, lacustrine silt, clay, sand, fluvial sand, and minor amounts of gravel. The maximum exposed thickness of intermediate drift is about 80 m (262 ft), in the Green River valley. The upper till is generally 3 to 10 m (10 to 35 ft) thick, but the lower till varies in thickness between 12 and 60 m (40 and 200 ft).

Puyallup Formation consists of thin beds of sand, silt, clay, and peat, and lies above the intermediate drift and below Salmon Springs Drift. The Puyallup Formation also contains volcanic ash and volcanic mudflow. The maximum known thickness of the Puyallup

Formation is 17 m (55 ft), at the mouth of the Green River valley. Along the Green and White River valleys, the formation probably is not more than 8 m (25 ft) thick.

The Salmon Springs Drift overlies the Puyallup deposits and underlies the Vashon Drift along the Green River, White River, and Duwamish Valleys. The Salmon Springs Drift consists chiefly of fluvial sand and gravel. The formation includes one or two till layers in the vicinity of Auburn. The thickness of the Salmon Springs Drift ranges from less than 15 m (50 ft) in the White River Valley to more than 122 m (400 ft) in the Auburn area.

Till or stratified drift of Vashon age lies at the surface of the drift plain in and around the project area. Vashon till forms the present surface of about half the drift plain, and probably extends under most other deposits that lie at the surface. Vashon till consists of an unstratified, nearly unsorted mixture of pebbles and cobbles in a clayey silt and sand matrix, and it contains scattered boulders as large as a meters across. The till is highly impermeable. It is highly variable in thickness; the maximum exposed thickness is about 21 m (70 ft), but the average thickness is about 6 m (20 ft).

Post Glacial Sedimentation

Post-glacial sedimentation began as the Puget lobe retreated to a point where it no longer contributed sediment to that locality. The post-glacial deposits may be partly late Pleistocene and mostly or entirely Holocene in age. Three principal types of deposits were formed, as briefly described below:

- Lacustrine deposits that consist of peat and lesser amounts of silt, clay, and sand occupy closed depressions and other poorly drained areas on the glacial drift plain and on the floors of the major valleys. On the valley floors, lacustrine sediments generally occur in broad, thin layers in flood-plain basins. The organic material in the peat bogs is chiefly woody, fibrous, sedimentary, and moss peat.

- Alluvium deposited on flood plains by the White, Green, and Cedar Rivers as they cut down into the drift plain is preserved locally on terraces along the present valley walls. Terrace alluvium along the Green River occurs in sheets of gravel and sand only about 5 to 10 m (16 to 33 ft) thick rather than in thick fill deposits. The terraces were formed by lateral swinging of the various rivers against their valley walls as they cut downward through the drift plain in post glacial time.

- Colluvium deposits are also found at the margins of the flood plains. These deposits consist chiefly of landslide debris but also include slope wash and even some alluvium on valley walls.

Detailed studies of the geology of the Kent Valley area are currently being performed by the State of Washington Department of Natural Resources (DNR, 1994). Geologic cross-sections developed by staff within the DNR indicate that the project area was at the south end of the ancient Duwamish embayment. The ancient delta for the Green River was located to the east, and the ancient Auburn delta was locate to the south. The bottom of the embayment at the project site was as much as 50 m (165 ft) below the current sea level.

Approximately 5,700 years ago, the area was covered by the Osceola Mudflow, which originated on the northeastern flank of Mount Rainier (DNR, 1994). In the project area the top of the mudflow is apparently located 30 to 60 m (100 to 200 ft) below the ground surface.

This mudflow is typically characterized as a dark gray, gravelly fine sand, silty sandy coarse gravel, and silty fine and coarse sand. Blowcounts from the SPT are often less than 10 blows per 0.3 m.

Subsequent to the flow the area was rapidly filled by the deposition of sediments from the White River. This led to a relatively thick accumulation of granular soils, with interlayers of silts and occasional deposits of clay in areas where still water occurred. Also, prevalent with the deposition was accumulations of wood fragments and similar organic deposits.

Seismicity

The project site is located in an area that has undergone earthquake loading in the past and can be expected to undergo earthquake loading in the future. The sources of and potential ground motions resulting from these seismic events are summarized below.

Source of Seismicity

Seismic events in the Puget Sound area are generally believed to result from three source mechanisms: (1) the very large magnitude (M 8½ +) Cascadia source off the coast of Washington, (2) the intraplate source (M7½) occurring 30 to 70 km (19 to 43 miles) beneath the Puget Sound, and (3) random crustal events (M6½ to 7) that could occur in the upper 30 km (19 miles) anywhere in the region.

Of the three sources mechanisms the highest risk to the project site results from the intraplate source mechanism, which is thought to be capable of producing ground motions as high as 0.5g (g = gravitational acceleration). The 1949 Olympia earthquake (M7.1) and the 1965 SeaTac earthquake (M6.5) are recent events associated with the intraplate fault mechanism. The recurrence interval for this source mechanism is usually cited as being from 35 to 110 years. Levels of acceleration from an intraplate event are expected to range from 0.15 to 0.3g.

Potential for Ground Motions

The peak firm-ground acceleration for the project site will be approximately 0.29g, based on WSDOT's recently developed map *Peak Ground Accelerations with a 10 Percent Probability of Exceedance in 50 Years*. It is understood that WSDOT prepared its map from the United States Geological Survey's (USGS) national acceleration map, which was developed by the USGS during the 1996 National Seismic Hazards Mapping Project. The USGS estimated peak firm-ground acceleration values by conducting probability studies for the three major source mechanisms affecting the Puget Sound Area.

Ground Motions for Liquefaction and Embankment Stability Assessments

Local site geology can result in amplification or attenuation of the firm-ground acceleration, depending on soil conditions at the site and the level of firm-ground acceleration. Considering the loose soils that exist between the ground surface and elevation -15 m (49 ft) and the level of firm-ground motion estimated from the WSDOT acceleration map, a small amount of ground motion amplification is expected to occur at the project site.

For this project, procedures given in the National Earthquake Hazards Reduction Program (NEHRP, 1995) report *Recommended Provisions for Seismic Regulations for New Buildings,* which now have been adopted within the 1997 edition of the Uniform Building Code (UBC), were used to determine the amount of ground motion modification. Based on the blowcounts recorded during the geotechnical exploration programs for the area, the site was classified as either a NEHRP Soil Profile Type S_D or Soil Profile Type S_E in Table 1.4.2.3a of Part 1 - Provisions. The corresponding site coefficient, F_a, for both of these soil profiles is 1.2, based on the estimated short-period acceleration value, A_a, of 0.29 at the site.

For the purposes of the liquefaction and embankment stability studies discussed later within each bridge section, the seismic coefficient is assumed to be equal to the product of the peak ground acceleration from WSDOT's map (0.29g) times the site coefficient (1.2), giving a design acceleration of 0.35g. The primary contributor to the seismic ground acceleration in the USGS probability study for the Puget Sound area is the intraplate source mechanism, which is normally assigned a magnitude of 7½. This magnitude was used when conducting liquefaction evaluations for the site.

Ground Motions for Bridge Design

It is understood from discussions with WSDOT's geotechnical engineers that the bridge design will follow procedures given in either the 1996 AASHTO Division I-A or the 1994 LRFD Bridge Design Specifications for the determination of seismic loading on the bridge. The following parameters are appropriate for these design approaches: (1) acceleration coefficient = 0.29, and (2) site coefficient = Type II.

Chapter 4 -- Bridge No. 167/112 N-E Ramp

Foundation design studies carried out for Bridge No. 167/112 N-E Ramp (N-E Ramp) included

- determining the axial capacity of driven piles and drilled shafts
- assigning soil properties for use in lateral response analyses of driven piles and drilled shafts, and
- estimating the allowable bearing pressures for the abutment footings.

In view of the potential for liquefaction of sands and silts prevalent in the upper 10 m (33 ft) of soil profile at the bridge site, the possible effects of liquefaction on the capacity of driven piles and drilled shafts, as well as the stability of abutment side and end slopes, were also evaluated. Methods used during and key results from these foundation capacity and liquefaction analyses are presented in this chapter.

Project Design Considerations

The N-E Ramp structure is located between 15th Avenue SW and SR-167, approximately 3.5 km (2 miles) north of 15th Avenue SW. The general location of the bridge is shown in Figure 1-1. This bridge will be widened on its east side by 3.7 m (12 ft) to provide an HOV lane.

Existing Structure

The N-E Ramp structure was constructed in the early 1970's from prestressed concrete. It is approximately 83 m (273 ft) in length and 8 m (26 ft) in width. The bridge is supported on two interior piers with each pier consisting of a single column. Columns have an exposed height of approximately 6 m (20 ft) and are located approximately 38 m (125 ft) apart. The ends of the bridge are supported by shallow strip footings located within the abutment fill.

The foundation for each column consists of a pile cap located approximately 2.5 m (8 ft) below the roadway surface. From the original design drawings, it appears that each pile cap is roughly 6 m (20 ft) in width and is assumed to be square in shape. Each pile cap is supported by creosote-treated timber piles with as many as 28 piles in a group. The estimated average length of the timber piles, based on the original design drawings, is approximately 11 m (36 ft). This results in the toe of the piles being located at an approximate elevation of 7 m (23 ft). The timber piles were required in the design drawings for the bridge to have a capacity of 360 kN (40 tons).

Approach fills for the bridge are approximately 8 m (26 ft) in height. The end of the abutment fill is sloped at 2H:1V (horizontal to vertical); side slopes on the east side of the approach fill range in steepness from 3H:1V to 2.5H:1V. A 1.5-m (5 ft) wide strip footing is located at each end of the bridge in the approach fill, approximately 3 m (10 ft) below the roadway surface. Design drawings indicate that the allowable bearing pressure on the footing is 290 kPa (3 tsf).

Site Conditions

The site is level except for the grade change to accommodate the approach fills for the bridge. Side slopes along the east side of the bridge, where widening will occur, are covered with brush and small trees. The areas at the base of the side slopes are undeveloped, and therefore should pose no significant obstructions to construction. A pond is located to the east of the south approach fill, but appears to be far enough from the toe of the abutment fill slope that the additional fill to accommodate the widening should not encroach on the pond.

Traffic on the bridge was only moderate during the period that field explorations were completed. The use of the roadway below the bridge seems to be heavier, as traffic goes from northbound SR-167 to eastbound SR-18. A large working area for the construction of the column foundation exists on the south side of the bridge; the working area for the north column is restricted and will likely require temporary realignment of the road during construction.

Subsurface Conditions

Seven test holes have been drilled and sampled for this bridge: four for the original bridge design and three as part of this task order. A piezometer was installed in one of the test holes completed for this widening project. Locations of the test holes are shown in Figure 4-1, which is located at the end of this chapter. Test hole logs based on past and the most recent explorations are included at the end of this report chapter. Limited numbers of laboratory grain-size tests were also completed as part of the widening project. Results of these tests are also included at the end of this chapter.

The geotechnical soil profile for this bridge consists of layered silts, sands, and gravels to the maximum depth of exploration, 37 m (120 ft). Figure 4-2 shows the soil profile that was developed from the test hole logs.

For the purposes of the foundation design studies, five primary soil layers are identified. The characteristics and approximate depths of these layers are summarized as follows, beginning at the ground surface:

- **Layer 1 -- Site Fill:** This material occurs from the ground surface to approximate elevation 15 (49 ft). It appears that approximately 3 m (10 ft) of the site soil were removed during original construction and were replaced with this material. The same material is used for the approach fills to the bridge. Generally the fill is a dense sandy gravel. From location to location and depth to depth, the amount of silt changes. The upper portions of this layer are above the water table; blowcounts from the SPT are normally greater than 20.

- **Layer 2 – Sandy Silt Layer:** This layer extends from approximate elevation 15 (49 ft) to approximate elevation 12 m (39 ft). The material is primarily fine silty sand and sandy silt. Blowcounts are sometimes less than 10. It is located below the water table.

- **Layer 3 -- Sandy Gravel Layer:** This layer occurs between approximate elevation 12 m (39 ft) and elevation 0 m (0 ft). The layer consists of a gravelly sand to sandy gravel. While blowcounts within the layer are typically above 25, lower blowcounts (e.g., less than 20) are recorded in the upper portions of the layer.

- **Layer 4 -- Loose Sand Layer:** This layer consists of 13 to 15 m (43 to 49 ft) of loose silty sand and gravelly sand. Traces of wood are noted in the test hole logs. Blowcounts range from 6 to 20 or more. Blowcounts in the top 5 m (16 ft) of this layer are often less than 15.

- **Layer 5 -- Dense Gravel Layer:** A dense gravel layer is encountered at approximate elevation -13 to -15 m (-43 to -49 ft). This layer is very consistent in the general area. Blowcounts from the SPT are in excess of 50 blows per 0.3 m (1 ft).

Several important features within the soil profile were identified from the test hole logs. First, low blowcounts occur within Layers 1, 2, and 4. While some of these low blowcounts appear to be caused by heave within the augers during drilling, at least some are thought to represent actual conditions. As discussed subsequently, the low blowcounts in Layers 1 and 2 lead to concerns about the susceptibility of these layers to liquefaction during a design earthquake. The low blowcounts in Layer 4 present concerns about the depths at which end bearing can be mobilized in driven piles or drilled shafts.

Another relevant observation during both the present and past exploration programs was the presence of scattered wood fragments and cobbles within the soil profile. A wood log with a diameter of 300 mm (12 in) was encountered in one of the 1970 test holes (H-1-70). A large cobble was also encountered in at least one of the test holes (H-3-70) at a depth of approximately 15 m (49 ft), requiring the use of dynamite to break the cobble.

Groundwater was measured at depths of 1.5 to 3 m (5 to 10 ft) below the ground surface. These depths correspond to approximate elevations of 18 to 20 m (60 to 66 ft). A design groundwater elevation of 20 m (66 ft) was used for static pile and drilled shaft analyses. For liquefaction analyses the groundwater elevation was assumed to be elevation 18 m (60 ft). This lower level for liquefaction analyses represented an expected long-term condition, while the higher elevation was used for pile and drilled shaft design to assure that adequate conservatism was incorporated in design for possible short-term loading conditions.

Engineering Soil Properties

Engineering properties were assigned for each of the primary soil layers to aid in subsequent foundation design computations. Various methods were used to assign these properties, including soil descriptions, blowcounts from the SPTs, and normal engineering judgment. These properties are best-estimated values, rather than lower bound. The fact that the values are best-estimates needs to be recognized as factors of safety are selected for determining the axial capacity of driven piles and drilled shafts. A summary of these properties is presented in Table 4-1.

Table 4-1. Summary of Estimated Soil Properties at N-E Ramp

Soil Layer No.	Moist Unit Weight (kN/m^3)	Saturated Unit Weight (kN/m^3)	Friction Angle
1	19.6	19.8	33
2	-	18.1	28
3	-	19.6	33
4	-	18.9	30
5	-	20.3	35

Liquefaction Susceptibility

Liquefaction assessments were conducted using the Seed-Idriss simplified blowcount procedure (Seed and Idriss, 1982) with a peak ground acceleration of 0.35g. As noted in Chapter 3 of this report, the peak firm-ground acceleration for the site is estimated to be 0.29g. This motion is expected to amplify by a factor of approximately 1.2, as the seismic wave propagates through the upper 30 m (100 ft) of soil profile, resulting in a design motion for the liquefaction and embankment stability studies of 0.35g.

In the liquefaction assessment blowcounts from both the 1997 and the previous exploration programs were used to estimate the cyclic resistance ratio (CRR) for the soil on a test hole by test hole basis. Blowcounts from all SPTs were adjusted to an energy of 60 percent. An energy ratio of 80 percent was used for the automatic hammer; all other blowcounts were assumed to be measured at an energy of 60 percent. Other CRR correction factors, including those for overburden, fines correction, and earthquake magnitude were consistent with the latest recommendations of Robertson and Wride (1997).

The liquefaction potential, which is equivalent to the factor of safety against the occurrence of liquefaction, at each test hole location was determined by comparing the computed value of CRR to the cyclic stress ratio (CSR) caused by the design earthquake. If the liquefaction potential was 1.1 or lower, the soil was identified as having a high potential for liquefaction during a design earthquake. A check was then made to determine if the material with a high liquefaction potential met the grain size and plasticity criteria identified by Seed and Idriss (1982) as being necessary for a material to be liquefiable. Locations of high liquefaction potential were then plotted on the soil profile for the E-N Ramp to determine the trend in liquefaction.

Based on the blowcount analyses it appears that liquefaction could develop between the groundwater location (i.e., elevation 18 m; 60 ft) and elevation 12 m (40 ft) at the N-E Ramp. This depth range encompasses the lower portion of Layer 1 and all of Layer 2. The potential for liquefaction is not, however, continuous within this elevation range. Rather, some of the blowcounts within the range suggest a low liquefaction potential, with the factors of safety against the occurrence of liquefaction in excess of 2. Individual points of liquefaction were then discounted if adjacent blowcounts were high, under the premise that re-distribution in porewater pressure would moderate the tendency for porewater pressure buildup. Likewise, blowcounts in areas where heave was specifically noted in the test hole log were also discounted.

From these interpretations, it was concluded that the soil between elevation 18 m (60 ft) and 15 m (49 ft) would be the most likely to liquefy on a relatively continuous basis; i.e., the entire layer would be liquefied at one time. Material between elevation 15 m (49 ft) and 12 m (40 ft) would undergo liquefaction on a more localized basis, with some zones of loose sands and silts liquefying but adjacent areas not liquefying.

Methods of Foundation Analyses

Foundation design studies were completed to determine the capacities of shallow and deep foundations that would likely be used during the widening project. The sizes for these foundations were provided by WSDOT's project manager. Approaches for the analyses were discussed with WSDOT prior to and during the analyses to confirm that the methods were generally consistent with WSDOT foundation design requirements.

Driven Pile Design

Axial pile capacities were determined for 460 and 610 mm (18 and 24 in) steel pipe piles. It was assumed that these piles would be driven with a closed end, and filled with concrete after driving. Analyses were conducted for these two pile sizes to determine the (1) axial capacity under static (service load) and seismic conditions, (2) the amount of settlement of a four-pile group under service loads, and (3) soil parameters for lateral pile capacity determination.

Static Axial Capacity Determination

Both compressive and uplift capacities of the piles were determined. The unified method of design (Fellenius, 1996) was used to estimate compressive and uplift capacities. Coefficients for β and N_t used during these analyses are given in Table 4-2. No limitations were place on the determination of side and end resistance when computing capacities. In some design methods a critical depth of 10 to 20 pile diameters is imposed, beyond which side friction and end resistance values do not increase. (e.g., DM-7, 1982). However, for the depths involved and based on discussions by Fellenius and Altaee (1995), there seems to be considerable question whether the critical depth concept is appropriate.

Table 4-2. Summary of Coefficients for Driven Pile Design at N-E Ramp

Layer No.	Static Conditions		Seismic Conditions	
	β	N_t	β	N_t
1	0.35	-	0.35	-
2	0.30	-	0.15	-
3	0.45	55	0.45	55
4	0.32	35	0.32	35
5	0.45	60	0.45	60

The uplift resistance was assumed to be 80 percent of the friction along the side of the pile in compressive loading. This reduction is consistent with WSDOT's standard practice.

Seismic Axial Capacity Determinations

Procedures used to estimate axial capacity under seismic loading differed from the method for estimating static capacity only in the (1) assigned β value for the lower portion of Layer 1 and all of Layer 2 and (2) the prescribed pile toe elevation. As discussed above, liquefaction is predicted at various depths in Layers 1 and 2 under a design earthquake, the consequence of which will be reduction in the side and end resistance for the pile. It was assumed for the seismic axial capacity determination that liquefaction would occur between elevation 18 and 12 (60 and 40 feet)

Throughout the liquefied zone, a reduced β value was used for side friction. The reduction in side resistance was introduced by using an undrained residual strength ratio (S_r/σ') equal to 0.15. This ratio was selected on the basis of information presented by Dobry and Baziar (1993) and in the draft proceedings from a 1997 National Science Foundation Workshop (NSF, 1997) dealing with the measurement of residual strengths in liquefied soil. A wide range of undrained strength ratios have been suggested for liquefied soil, and some individuals contend that the residual strength is not proportional to the effective overburden pressure. Considering the differences of opinion that currently exist, a check was also performed using the relationship between blowcount and residual strength suggested by Seed and Harder (1990). An undrained strength ratio of 0.15 results in undrained strengths that are not inconsistent with the range determined from the Seed and Harder relationship.

It was further decided that the toe of the pile should be located below the zone with a high risk of liquefaction (i.e., 18 to 12 m; 60 to 40 ft) to minimize the potential for excessive pile settlement during a design seismic event. No adjustments were made for potential buildup in porewater pressure below the liquefied zone. It was assumed that sufficient conservatism had been introduced by establishing the maximum toe elevation below the maximum predicted depth of liquefaction.

This approach to liquefaction was expected to be conservative. The actual effects of the assumption regarding side friction on compressive and uplift capacity are not significant, as the side resistance within this depth interval is relatively small, even under static conditions.

Settlement Estimates for Static Loading

Settlement estimates were made assuming that four piles would be required to support the pile cap for the column. The four-pile configuration was selected primarily to provide increased lateral stiffness, in the event that loss in soil strength occurs in part of Layer 1 and in Layer 2 due to liquefaction, as predicted. It was also assumed that the four piles would be spaced at 2 ½ to 3 diameters.

An equivalent footing approach was taken in estimating settlements. The size of the footing was defined by the perimeter of the pile group. This footing was located at the neutral plane of a single pile, where the neutral plane was defined as the point at which the side friction for the pile equals the service load. A 2V:1H stress distribution was assumed below the footing.

Soil Parameters for Lateral Pile Loading

Procedures used to determine soil parameters for lateral load generally followed recommendations of Reese and others (e.g., Reese and Wang, 1989a). Modulus of subgrade reaction values were based on information presented in Lam and Martin (1986), which gives modulus of subgrade reaction values as a function of relative density for sands located above and below the water table. These parameters are appropriate for use in the computer programs LPILE and COM624.

For seismic loading the resistance of Layer 1 and Layer 2 was reduced to account for the likelihood of liquefaction under a design earthquake. While liquefaction could occur between elevation 18 and 12 m (60 and 40 ft), the lower portion of Layer 1, which makes up the upper 3 m (107 ft) of the liquefiable zone, was considered most vulnerable. Within this layer a fully liquefied condition was assumed. The average corrected blowcount, $(N_1)_{60}$, for this layer was approximately 12, resulting in a β of 0.15 based on NSF (1997) or a residual strength of 12 kPa (250 psf) based on the lower bound of the relationship between residual strength and corrected SPT value given by Marcuson et al. (1990). Below approximate elevation 15 the liquefied zone was assigned a friction angle midway between the liquefied and nonliquefied values. The basis for this was that random locations of liquefaction were predicted potentially between elevations 15 and 12 m (49 and 40 ft). However, other locations within the same depth range did not liquefy. Realizing this, it was reasoned that some loss in lateral support capacity would occur, but more resistance would exist than a fully liquefied state.

Pile-group reduction factors were also defined to account for interaction between piles if the piles are closely spaced, as expected. The reduction factor will depend on the selected spacing ratio (i.e., ratio of center-to-center pile spacing to pile diameter). Significant differences in opinion currently exist within the profession regarding the form and amount of the reduction to apply. Based on a recent survey of state departments of transportation (Brown et al., 1998), it was found that reduction factors given in references such as DM-7 (1982), the Canadian Foundation Engineering Manual (1985), and even the Federal Highways Administration (FHWA) Manual *Design and Construction of Driven Piles* (GRL, 1996) are generally viewed as resulting in too much reduction in stiffness. The p-multiplier procedure (e.g., Brown and Bollmann, 1996) is currently thought to provide the most realistic representation of group effects, in the absence of dynamic analyses such as given in WSDOT's Design Manual *Foundation Stiffness Under Seismic Loadings* (GeoSpectra, 1997).

Drilled Shaft Design

Axial capacities of two drilled shafts, with diameters of 1.83 m (6 ft) and 2.44 m (8 ft), were determined. It was assumed that a steel casing would be used during installation of these shafts, but that the casing would be removed as the concrete is placed. Analyses were conducted for each shaft diameter to determine (1) the axial capacity under static (service load) and seismic conditions, (2) the possible settlement of the shaft under service loads, and (3) soil parameters for lateral shaft capacity determination.

Static Axial Capacity Determination

The static capacity of the shaft involved determination of side resistance, end bearing, and uplift resistance. Procedures suggested by the FHWA Manual *Drilled Shafts* (Reese and O'Neill, 1988) were generally followed when determining capacity. In this approach the end bearing of the shaft is determined from the product of the uncorrected blowcount (N) times a factor of 57.5 in kPa (or N X 0.6 in tsf), and the side friction for cohesionless soil is based on a computed β value.

Procedures used in the estimate of shaft side resistance deviated from recommendations given in the FHWA manual in one important area. When determining β values, the equation recommended in the FHWA manual was not followed. During a progress review meeting with WSDOT's geotechnical engineers, it was decided that the β values determined from the equation in the FHWA manual were too high in the upper layers of soil and possibly too low in the lower layers. To obtain what were considered to be more representative β values for the soil conditions at the site and the likely construction methods, β was defined as the product of a lateral earth pressure coefficient (k) and the tangent of the interface friction angle.

Shaft capacity computations were performed using the Ensoft computer program SHAFT1 (Reese and Wang, 1989b). This program computes shaft side and end resistance every 0.3 m (1 ft) throughout the depth of interest. Input to the program includes β and blowcounts for each layer. The values of β and the average N values used for the shaft capacities analyses are summarized in Table 4-3. As with the driven piles, the uplift capacity was assumed to be 80 percent of the side friction for the pile in compression.

Table 4-3. Summary of Coefficients for Drilled Shaft Design at N-E Ramp

Layer No.	Static Conditions		Seismic Conditions	
	β	N	β	N
1a 1b	0.33 0.33	10 10	0.33 0.15	- -
2	0.27	8	0.15	-
3	0.42	32	0.42	32
4	0.29	10	0.29	10
5	0.54	70	0.54	70

Seismic Axial Capacity Determinations

Procedures used to estimate the axial capacity of the shaft under seismic loading differed from the method for estimating static capacity only in the assigned β value for the lower portion of Layer 1 and all of Layer 2. As discussed previously for driven piles, liquefaction is predicted at various depths in these layers under a design earthquake, the consequence of which is reduction is the strength of the layer. It was assumed that the β value would be reduced to 0.15 between elevations 18 and 12 m (60 and 40 ft). The rationale for the selection of β of 0.15 is the same as that given for driven piles. Also similar to the driven pile, it was concluded that the shaft should be located below the maximum predicted depth of liquefaction.

Settlement Estimates for Static Loading

Settlement estimates were made assuming that a single shaft would support each column. An equivalent footing approach was taken in estimating settlements. The size of the footing was defined by the perimeter of the shaft. This footing was located at the neutral plane of the shaft. As noted before, the neutral plane was defined as the point at which the side friction for the shaft equals the service load. A 2V:1H stress distribution was assumed below the equivalent footing.

Soil Parameter for Lateral Pile Loading

Procedures used to determine soil parameters for lateral load were the same as those used for driven piles. After discussions with WSDOT's geotechnical engineers, it was decided that no adjustment factors would be given to account for the potential effects of shaft diameters greater than 0.6 m (2 ft), as has recently been suggested in some studies (e.g., ATC, 1996). These parameter are appropriate for use in the computer programs LPILE and COM624.

As with the driven piles, the resistance of the soil between elevation 18 and 12 meters (60 and 40 feet) was reduced to account for the likelihood of liquefaction under a design earthquake. While liquefaction could occur throughout the elevation range, the upper 3 m (10 ft) were considered most vulnerable. Within this layer a fully liquefied condition, with a residual strength of 12 kPa (250 psf), was assumed. The lower portion of the range was assigned a friction angle midway between the liquefied and nonliquefied values. The basis for this was the same as discussed previously for driven piles.

Abutment Design

To facilitate the widening, it will be necessary to increase the width of the embankment side slopes by approximately 4 m (13 ft). Abutment footings will also have to be constructed in the approach fill to support the new bridge width. In the case of the abutment fill, analyses were performed to determine the stability of the new side slopes and end slopes under static and seismic loading. For the abutment footings, it was necessary to determine allowable bearing pressures and strain compatible dynamic soil properties for the footing. Procedures used to evaluate these requirements are summarized below.

Abutment Stability

The stability of the abutment fill was determined by conducting stability analyses using the computer program PCSTABL (Siegel, 1975). For these analyses the groundwater was assumed to be located at elevation 18 m (60 ft), which is roughly 3 m below the existing ground surface. The end slope of the embankment was assumed to be 2H:1V, which was similar to the end slope and somewhat steeper than the side slopes. Properties of the embankment material and underlying soils were as defined previously within the discussion of Engineering Soil Properties.

For the seismic case, pseudo static analyses were conducted using PCSTABL. In this approach the seismic coefficient was varied until a factor of safety approximately equal to 1.0 was defined. Properties were similar to those used for the static analyses, except that the lower portion of Layer 1 was assigned a residual strength equal to 0.15 times the effective overburden pressure (i.e., $S_r = 0.15\sigma'$). The basis for the residual strength determination was presented previously in the discussion of Driven Pile Design. The lower portion of Layer 1 was used to constrain the depth of the failure surface to the zone where continuous liquefaction was expected.

Estimates of deformation during the seismic event were made using the Newmark simplified method. With this method, an approximate estimate of deformation can be obtained from published relationships between the predicted deformation and the ratio of yield acceleration to peak acceleration.

Allowable Footing Pressures and Dynamic Properties

Each end of the existing bridge is supported on an abutment wall that is supported on a 1.5-m (5 ft) wide strip footing extending across the complete width of the bridge. This footing is located approximately 3 m (10 ft) below the roadway surface. It is anticipated that a similar size footing at the same depth will be used for the widening. Allowable bearing pressures for this footing were determined using conventional bearing capacity theory with allowances for the sloping face of the end abutment. It is understood that the lateral earth pressures for the abutment wall will be based on WSDOT's standard wall design.

Shear modulus, material damping, and Poisson's ratio values were estimated based on recommendations given in the FHWA Manual *Seismic Design of Bridge Foundations* (Lam and Martin, 1986). For these parameter determinations the low-strain shear modulus was selected on the basis of average blowcounts recorded during the SPTs within one footing width below the planned footing elevation. An average shearing strain of 0.02 to 0.2 percent was used to adjust for the level of shearing strain expected during a design event.

Recommendations

This presentation of recommendations is separated into two sections. The first covers the foundation systems, and the second involves construction considerations. While the discussion of construction is limited, recommendations given for design of the foundation systems are dependent on the methods used and observations made during construction. For this reason it is critical that any changes in either site conditions encountered during construction or procedures used during construction be brought to the attention of CH2M HILL in order that the following foundation recommendations can be confirmed for the observed conditions or methods.

Foundations

The methods of analyses described in the preceding section were used to develop geotechnical recommendations for design of driven pile and drilled shaft foundations, abutment footings, and abutment slopes under static and seismic loading conditions. These recommendations are based on best estimates of soil properties. Appropriate consideration should be given to the possibility of different soil properties and soil behavior during selection of factors of safety.

Driven Piles and Drilled Shafts -- Static Loading

The interior columns for the bridge can be supported using either driven piles or drilled shafts

Axial Capacity: Figures 4-3 through 4-6 present **ultimate** axial capacity versus depth plots for each pile and shaft size. It is emphasized that these capacities are ultimate values; they have not been reduced with factors of safety. The maximum ultimate capacity for driven piles is limited to 4,500 kN (500 tons) to keep the ultimate capacity within the range of applicability of the dynamic formula in Section 6-05 of WSDOT's Standard Specifications.

Allowable values can be determined by applying a factor of safety to the capacities given in Figures 4-3 through 4-6. Table 4-4 provides recommended factors of safety for design. As shown in this table, the factor of safety should be selected on the basis of the type of field monitoring that is done before or during pile or shaft installation. It is understood that WSDOT normally will monitor pile drivability or shaft construction; however, if test piles are driven or a static load test were performed, lower factors of safety would be appropriate.

Table 4-4. Recommended Factors of Safety at N-E Ramp

Field Confirmation	Driven Piles		Drilled Shafts	
	Compressive Loading	Uplift Loading	Compressive Loading	Uplift Loading
None	3	3	4	4
Standard WSDOT	2.5	1.5	2.5	1.5
Test Piles/PDA	2.25	1.4	-	-
Static Load Test	2.0	1.3	2.0	1.3

Minimum and maximum pile or shaft toe elevations should be used with Figures 4-3 through 4-6 to assure development of the required capacities and to limit settlements. Table 4-5 provides a summary of the minimum and maximum toe elevations for the bearing layers. These elevations were established (1) to avoid locating the toe of the driven pile or drilled shaft in what was thought to be a more compressible material (e.g., Layer 2), (2) to locate the toe of the shaft or driven pile below the maximum anticipated depth of liquefaction, and (3) in the case of drilled shafts to limit construction to depths that WSDOT believes can be achieved without great risk of problems. Layers that should not be used for end bearing due to soil type or liquefaction potential are identified with "NA", meaning not appropriate. For any layer, a four-pile group founded between the minimum and maximum toe elevations is expected to develop the capacities given in Figure 4-3 through 4-6 with settlements under service loading of less than 25 mm (1 in).

Table 4-5. Summary of Minimum and Maximum Toe Elevations at N-E Ramp

Layer Number	Driven Piles		Drilled Shafts	
	Minimum Elev. (m)	Maximum Elev. (m)	Minimum Elev. (m)	Maximum Elev. (m)
1	NA	NA	NA	NA
2	NA	NA	NA	NA
3	12	2	12	3
4	-3	NR*	-3	-10

* Not Restricted

Lateral Capacity: Soil properties that should be used for non-seismic lateral pile capacity analyses are summarized in the LPILE/COM624 forms given in Table 4-6. The elevation of the top of the first layer should be the bottom of the pile cap for driven piles or 1.5 m (5 ft) below the ground surface at the shaft location.

Table 4-6. LPILE/COM624 Parameters for Service Loading at N-E Ramp

Layer No.	Type of Soil	Layer Elevation				Effective Unit Weight		Cohesion		Friction Angle	Coefficient . of Subgrade. Reaction		Soil Type
		Upper		Lower									
		(m)	(ft.)	(m)	(ft.)	(kN/m³)	(pcf)	(kPa)	(psf)	(degr.)	(MN/m³)	(pci)	
1a	Sand	-	-	18	60	19.6	125	0	0	33	24	90	4
1b	Sand	18	60	15	49	9.8	63	0	0	30	9	35	4
2	Silt	15	49	12	39	8.3	53	0	0	30	9	35	4
3	Sand w/ gravel	12	39	0	0	9.1	58	0	0	33	16	60	4
4	Sand w/ silt & gravel	0	0	-13	-43	9.1	58	0	0	30	9	35	4
6	Sandy Gravel	-13	-43	-	-	10.5	67	0	0	35	23	85	4

Group reduction factors should be applied if driven piles have spacing ratios of less than five diameters. The group reduction factors given in the following table were developed from Brown and Bollmann (1996). These values apply to the average stiffness of the pile group.

Table 4-7. Group Efficiency Factors for Driven Piles at N-E Ramp

Row Spacing	3-Pile Group	4-Pile Group	6-Pile Group
3 diameters	0.75	0.65	0.60
4 diameters	0.90	0.85	0.80
5 diameters	1.0	1.0	0.95

Driven Piles and Drilled Shafts -- Seismic Loading

Figures 4-7 through 4-10 present capacity versus depth plots for each pile and shaft size for seismic loading. These plots can be used with seismic loads to confirm that adequate axial capacity still exists when liquefaction occurs in the upper two soil layers. In view of the conservative approach used in considering liquefaction for the axial capacity determinations, factors of safety of 1.0 and 1.3 should be adequate for driven piles and drilled shafts, respectively, during a seismic event. In view of the high liquefaction potential in the lower portion of Layers 1 and all of Layer 2, a minimum toe elevation is established at elevation 12 m (40 ft).

The pile or shaft foundation system could settle during the seismic event. This settlement is expected to result from two sources: (1) the added pile or shaft loads resulting from the inertial response of the structure; and (2) densification of the upper portions of Layer 4. Settle from added bridge loads is expected to be small. Settlement from the densification of loose materials in the upper portion of Layer 4 could result in up to 50 mm (2 in) of settlement within Layer 4. Driven piles or drilled shafts founded above Layer 4 could settlement this amount. Similar amounts of settlement would also be expected to occur at the approach fills. If the driven piles or drilled shafts are founded in Layer 5, then settlement of the interior piers could occur due to drag loads as loose soils densify; however, this settlement is expected to be small. Settlement would still occur at the approach fills, resulting in differential movements between Pier 1 and Pier 2 and between Pier 3 and Pier 4. The amount of this differential movement could be as much as 50 mm (2 in).

Soil properties that should be used for lateral pile capacity analyses during seismic loading are summarized in Table 4-8. Group adjustment factors discussed above for static loading should be applied. Inasmuch as the phasing between liquefaction and maximum inertial forces on the bridge structure is difficult to predict, it is recommended that seismic analyses include lateral capacity evaluations for two cases: (1) a nonliquefied case, which is equivalent to the static case (Tables 4-6), and (2) the seismic case given below. Design should be based on the more critical of the two.

Table 4-8. LPILE/COM624 Parameters for Seismic Loading at N-E Ramp

Layer No.	Type of Soil	Layer Elevation				Effective Unit Weight		Cohesion		Friction Angle	Coefficient of Subgrade Reaction		Soil Type
		Upper		Lower									
		(m)	(ft.)	(m)	(ft.)	(kN/m³)	(pcf)	(kPa)	(psf)	(degr.)	(MN/m³)	(pci)	
1a	Sand	-	-	18	60	19.6	125	0	0	33	24	90	4
1b	Sand	18	60	15	49	9.8	63	12*	250	-	-	-	1
2	Silt	15	49	12	39	9.1	53	0	0	21	5	18	4
3	Sand w/ gravel	12	39	0	0	9.8	63	0	0	33	16	60	4
4	Sand w/ silt & gravel	0	0	-13	-43	9.1	58	0	0	30	9	35	4
5	Sandy Gravel	-13	-43	-	-	10.5	67	0	0	35	23	85	4

*Note: For Layer 1b, assume $\varepsilon_{50} = 0.02$ mm/mm.

Abutment Footings

The abutment footing should be designed for an allowable bearing pressure of 290 kPa (3 tsf). With this loading the settlements are expected to be less than 25 mm (1 in). Roughly half of the settlements is expected to occur during construction of the footing and abutment wall. For seismic loading (i.e., Load Case 7) the allowable pressure on the abutment footing can be increased by a factor of 2.

Shear modulus, material damping, and Poisson's ratio properties given in Table 4-9 are recommended for determining stiffness values for seismic design. These properties were developed using a shear wave velocity of 250 mps (820 fps), which results in a low-strain shear modulus of approximately 120 MPa (2,500 ksf).

Table 4-9. Dynamic Soil Properties for Abutment Footing at N-E Ramp

Mode of Vibration	Shearing Strain = 0.02%	Shearing Strain = 0.2%
Shear Modulus	80 MPa (1,700 ksf)	30 MPa (630 ksf)
Material Damping	5%	12%
Poisson's Ratio	0.35	0.35

In the event that future design studies determine that strip footings cannot be used, because of the available room or for whatever other reason, it would be possible to use drilled shafts or driven piles to support the abutment wall. Axial and lateral capacity information presented in this chapter for the closest pier can be used for drilled shaft and driven pile designs at the abutment should a spread footing not be feasible.

Embankment Slopes

The side slopes in the widened area should not exceed 2.5H:1V, which is the maximum existing side slope. End slopes should not exceed 2H:1V, which is also the existing slope steepness. For these slope angles the factor of safety for static loading will be greater than 1.5.

During a design seismic event, deformations of the end slopes and side slopes could occur. The amount of deformation is estimated to be less than 0.3 m (1 foot). Deformations at the end slope could impose loads on the foundations for the columns. These loads would be imposed on the existing foundations, as well as the foundations for the widening project. In the event that at some future date a seismic retrofit is performed for the widened bridge, the retrofit should consider the potential effects of these additional loads on the foundation system. These effects could be evaluated by conducting lateral analyses of pile or shaft foundations with an imposed load from the moving soil. If the level of deformations cannot be tolerated, various ground improvement methods could be considered as part of the overall retrofit program.

Construction

Construction of the foundations for the widening project requires consideration of a number of issues related to both quality control and construction methods. A number of these issues specific to this project site are summarized below. In most cases the contractor should be made aware of these issues or requirements at the time of bidding.

Driven Piles

The primary construction issues and requirements associated with the use of driven piles are as follows:

- The potential for wood and cobbles exists throughout the soil profile, and particularly in Layers 3 to 4. While these conditions were not widespread, sufficient cases were noted during the drilling of test holes to warrant consideration during the contracting of pile installation. Pile driving contractors should be advised of this possibility within the special provisions.

- In recognition of the uncertainties of axial pile capacity, test piles should be installed prior to establishing pile order lengths. These test piles should be of the same size and should be driven with the same equipment as will be used during construction. The recommended numbers and locations of the test piles are given in the following table.

Table 4-10. Recommended Test Pile Program at N-E Ramp

Bridge	Pier Number	Number of Tests
N-E Ramp	2 3	1 1

- Groundwater could be located within 1.5 to 3 m (5 to 10 ft) of the ground surface. Depending on the location of the bottom of the pile cap, excavations below the ground water elevation could be required. The permeability of Layer 1, in which the pile cap would likely be located, is expected to be high. With this high permeability, it would be essential for the contractor to have identified procedures for handling excess water in the excavation. If winter construction is anticipated, seals may be required to control water. If summer construction occurs, dewatering systems may be sufficient to control water.

- Site access will be very restricted for the northern of the two piers at this bridge. It will likely require lane closures and, possibly, rerouting of traffic.

Drilled Shafts

The primary construction issues and requirements for drilled shaft will be as follows:

- The water table is very high for the site. This will necessitate the use of steel casing from the ground surface to the maximum depth of construction. It is critical that the casing be removed during placement of concrete, as friction values used for shaft capacity design are based on a soil-concrete interface and not a soil-steel interface. If the casing cannot be removed, shaft side resistance could decrease by as much as 50 percent.

- Shaft lengths could be up to 30 m (100 ft) in length to meet fixity requirements during seismic events. For these lengths quality control during placement of concrete will be critical. Realizing the potential consequences of poor quality control, WSDOT should plan to conduct sonic crosshole logging in each shaft following construction.

- Site access will be very restricted for the northern of the two piers at this bridge. It will likely require lane closures and, possibly, rerouting of traffic.

Abutment Footing

The primary issues related to the construction of the abutment footing are as follows:

- It will likely be necessary to use sheet piling to support the existing abutment fill during excavation for and construction of the new footing. The depth of excavation for the footing will be 3 to 4 m (10 to 13 ft), if the footing is similar in size to the existing footing (i.e., 1.5 m; 5 ft). However, if a wider footing is needed to meet slope-setback requirements, deeper excavations may be required.

- In the event that the new footing is located below the existing footing, special care will be required to avoid loss of footing support for the existing footing during construction. Sheet piling or other support methods are available to provide this support. However, it should be made clear in the special provisions that support of the existing footing must be maintained. It would be desirable to survey the location of the abutment wall before construction to be able to quantify any movement that does occur.

- Considering the potential for layers of siltier materials at the base of the planned footing excavation, the footing excavation should be carried to at least 0.3 m (1 ft) below the planned base of the footing. Crushed ballast should be compacted to the base of the footing to assure good drainage and high base friction.

Abutment Slopes

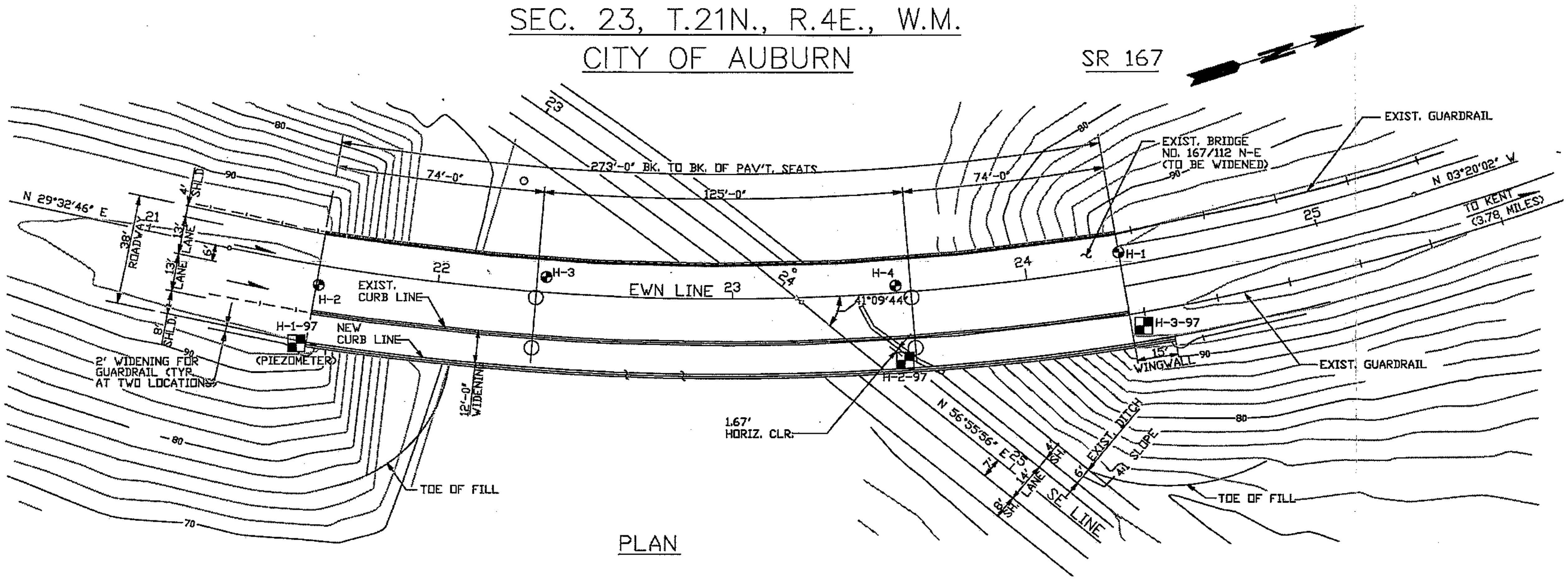
The primary construction issues and requirements related to the abutment slopes are as follows:

- The new side slope fill should be keyed into the existing fill by cutting benches into the existing embankment, as specified in WSDOT's standard specifications.
- Concrete slope protection matching the existing slope protection should be used to prevent ravelling of embankment materials beneath the bridge.

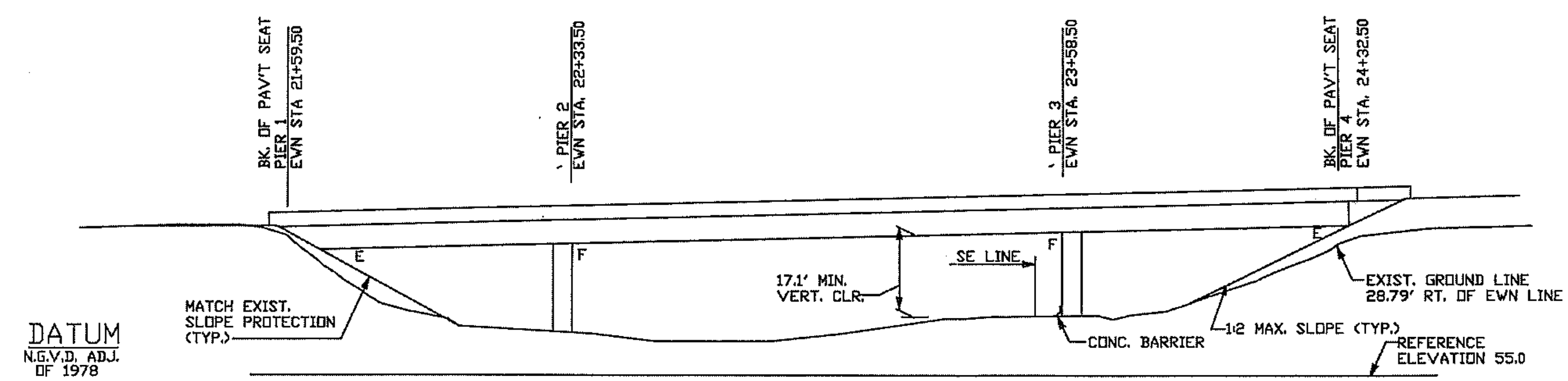
116184.G4 - N/E PLAN - 01/09/98 112new1.dwg

SEC. 23, T.21N., R.4E., W.M.
CITY OF AUBURN

SR 167



PLAN



DATUM
N.G.V.D. ADJ. OF 1978

ELEVATION

NOTES

1. DRAWING ADAPTED FROM PRELIMINARY BRIDGE PLAN DATED OCTOBER 1997. FINAL PLANS MAY VARY.
2. 1 FOOT = 0.305 METERS

SYMBOLS

- EXISTING WSDOT TEST HOLE
- 1997 WSDOT TEST HOLE
- △ 1991 TERRA TEST HOLE

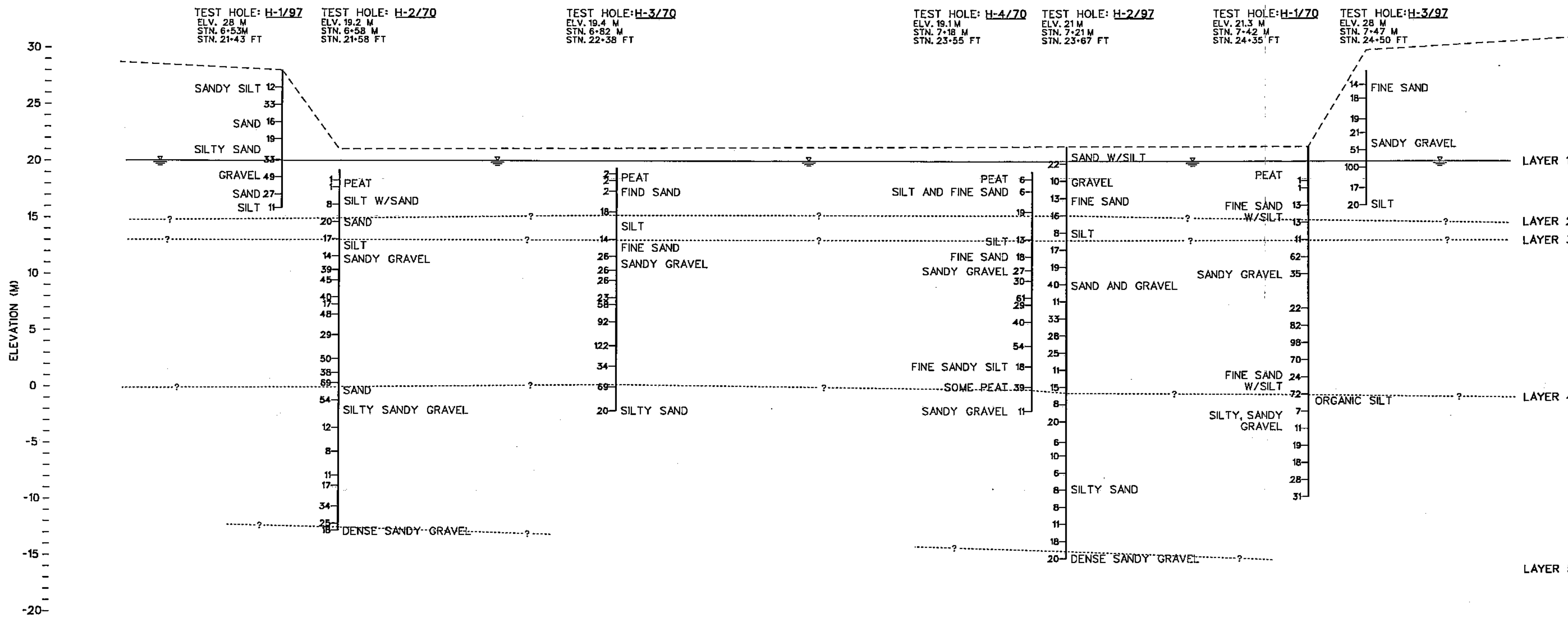
0 40 80
SCALE

Washington State Department of Transportation

CH2MHILL

FIGURE 4-1
N-E RAMP WIDENING
TEST HOLE LOCATIONS

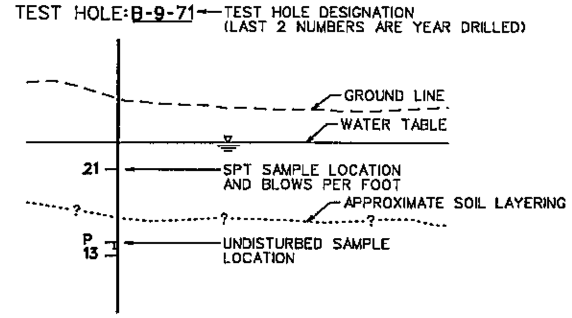
C.S. 1765, PROJECT NO. OL-2305, NORTHWEST REGION, 15TH ST. SW TO 15TH NW, HOV LANE - STAGE 3, SR 167, BR. NO. 167/112 N-E RAMP WIDEN.



Notes:

1. Soil layering is based on interpretations from soil test hole logs and engineering judgment. Actual conditions within and between test holes could differ from those indicated.
2. Water table elevation based on maximum estimated conditions. Actual elevation could be as much as 3 meters below identified elevation.

Legend Key:



0 5 10 15
SCALE: 1"=10M

Figure 4-2

Soil Profile For Bridge No. 167/112 N-E Ramp Geotechnical Report

SR-167, OL-2305
15th Avenue SW To 15th Avenue NW
HOV Widening Project

CH2MHILL

116184.G4 FILE NAME: 11618403.dlv 10-FEB-1998 07:38:39

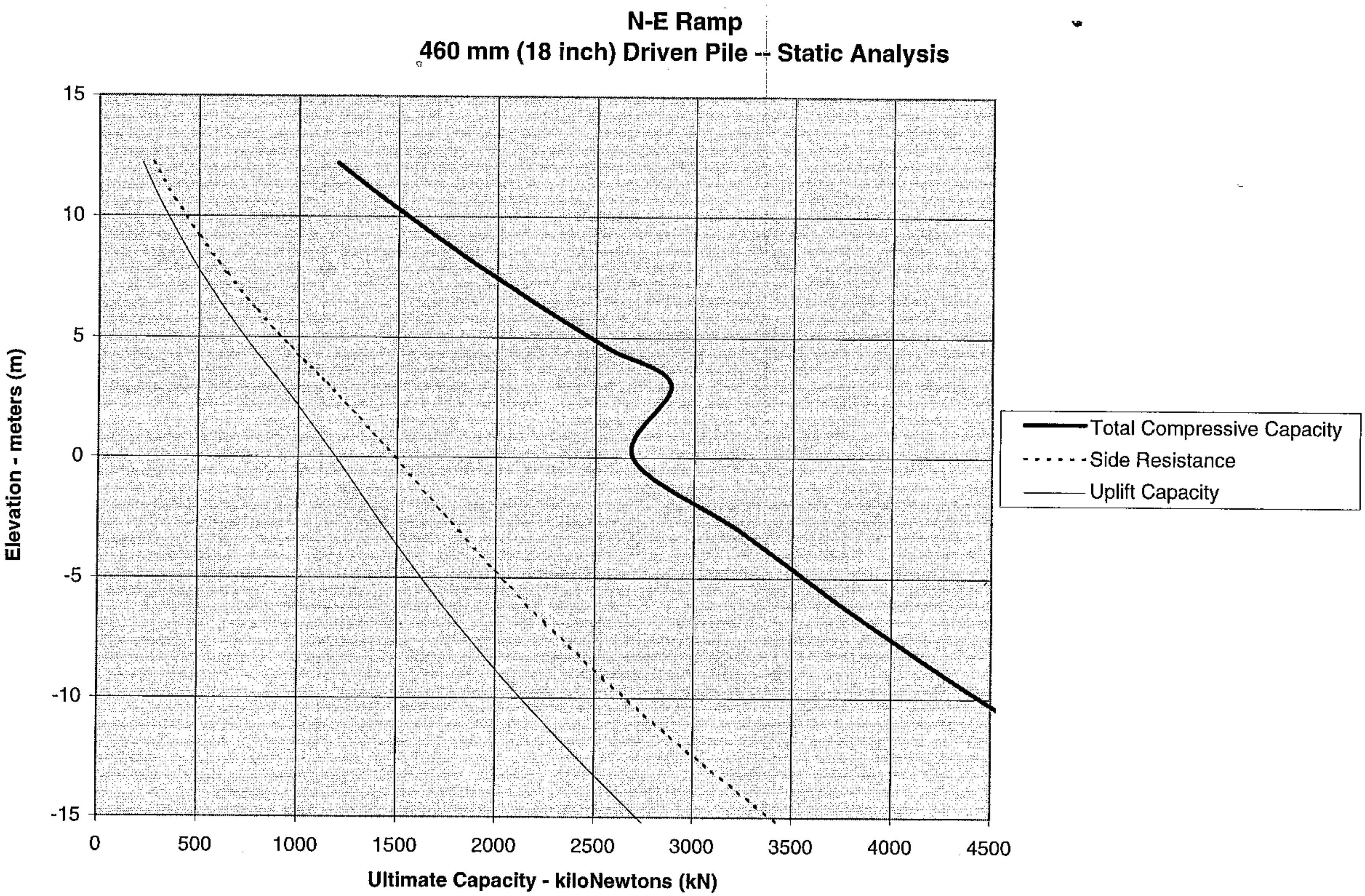


Figure 4-3. Ultimate Pile Capacity for 460 mm (18 inch) Driven, Closed-End Steel Piles at N-E Ramp – Static Analysis

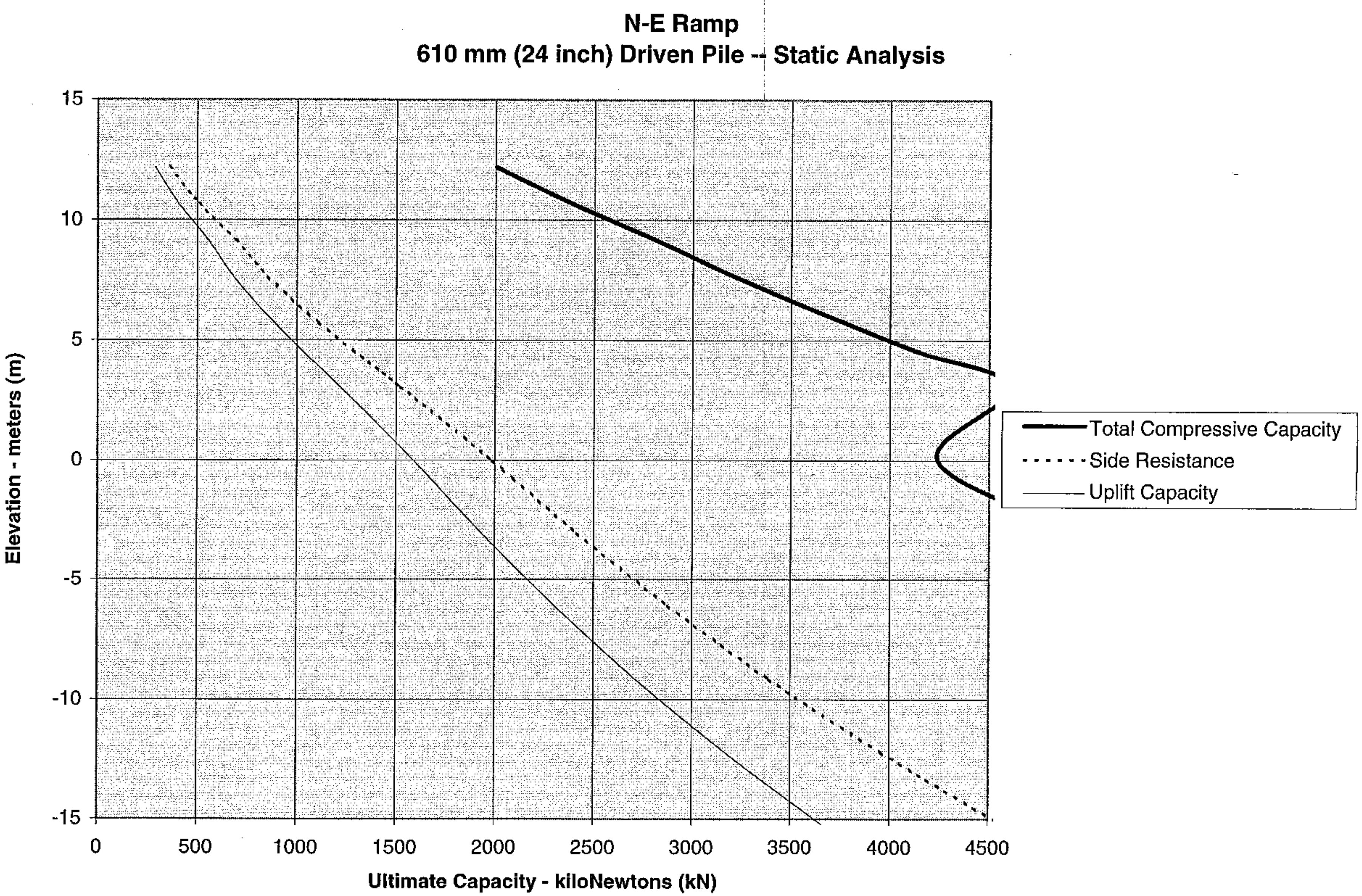


Figure 4-4. Ultimate Pile Capacity for 610 mm (24 inch) Driven, Closed-End Steel Piles at N-E Ramp – Static Analysis

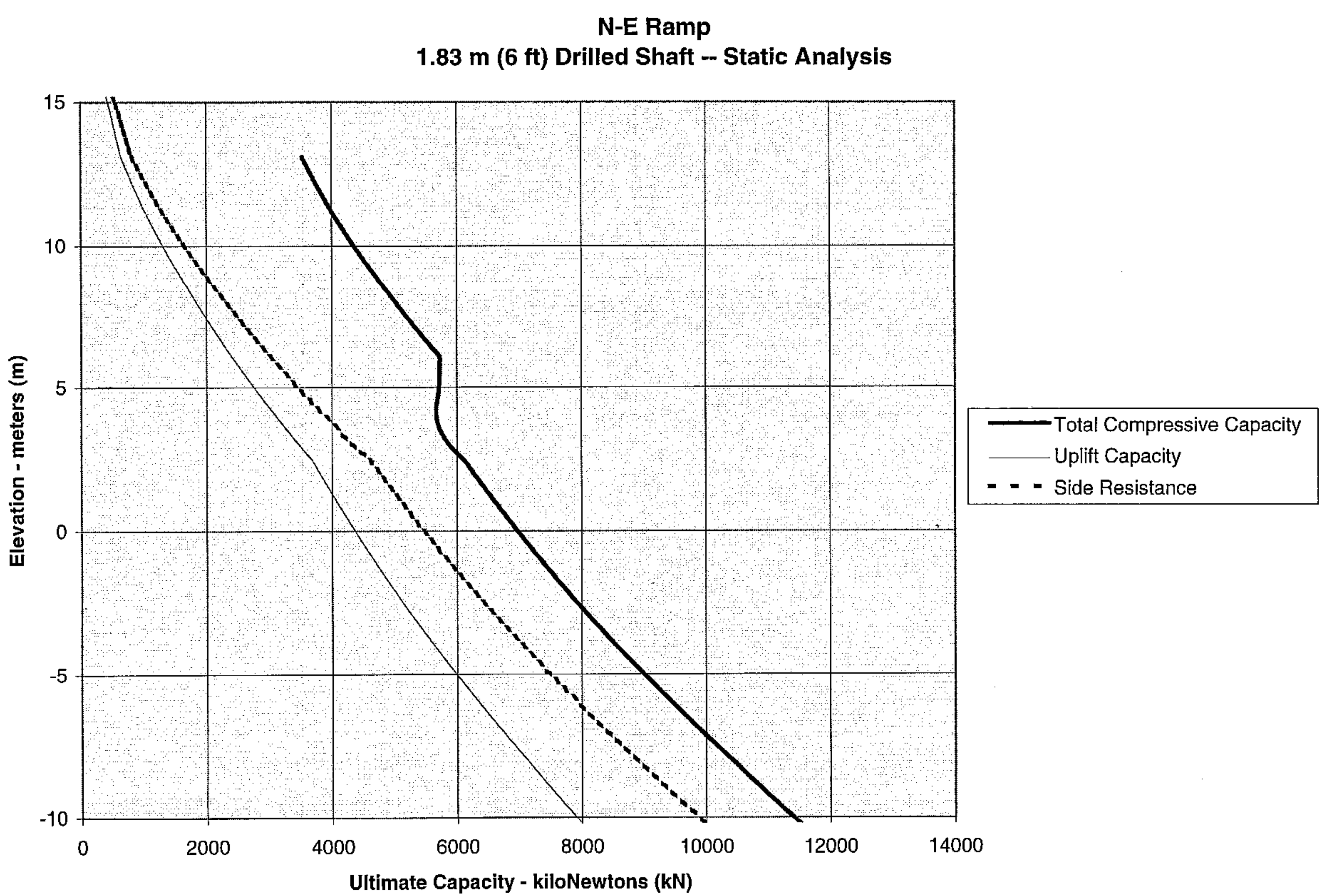


Figure 4-5. Ultimate Drilled Shaft Capacity for 1.83 m (6 ft) at N-E Ramp – Static Analysis

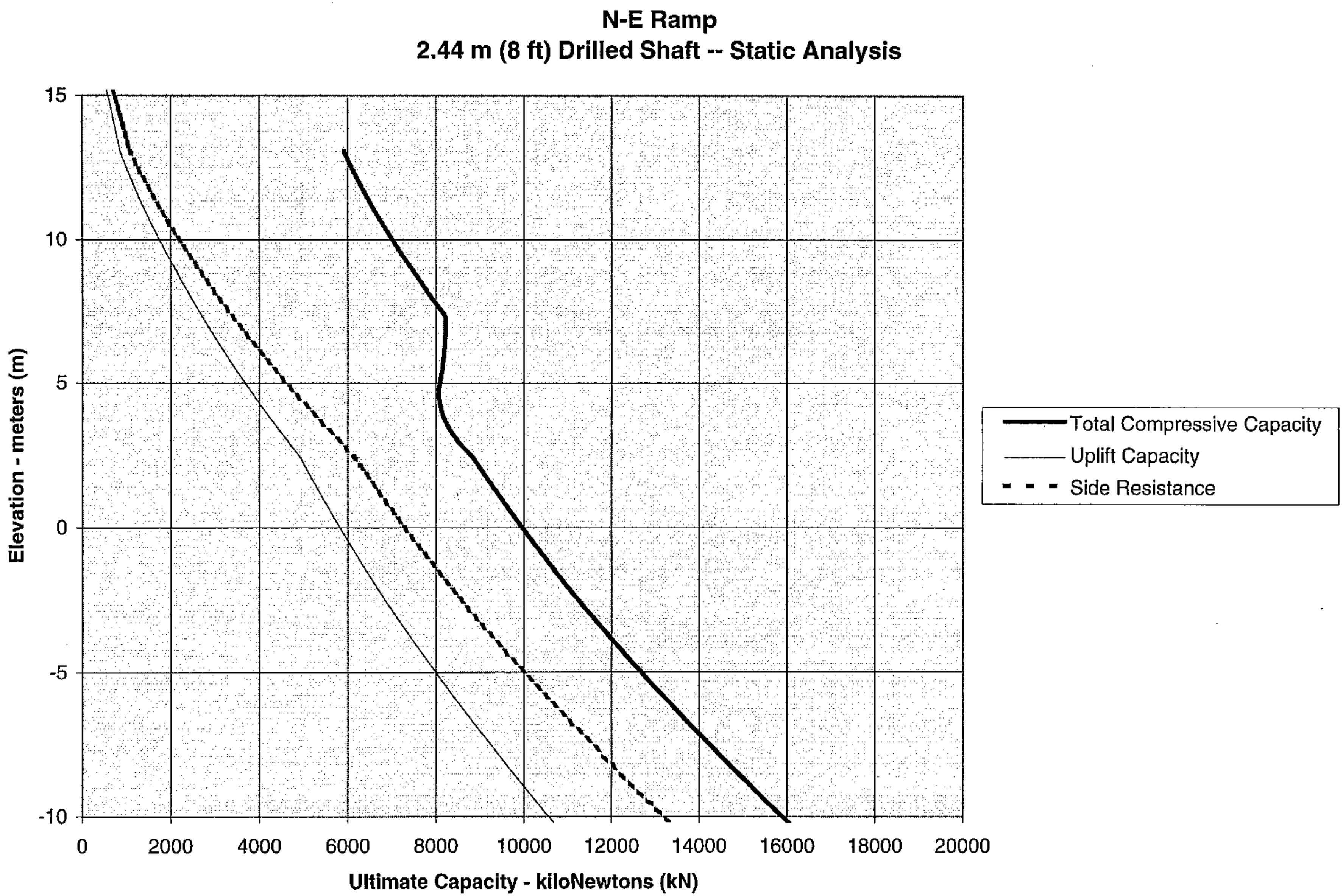


Figure 4-6. Ultimate Drilled Shaft Capacity for 2.44 m (8 ft) at N-E Ramp – Static Analysis

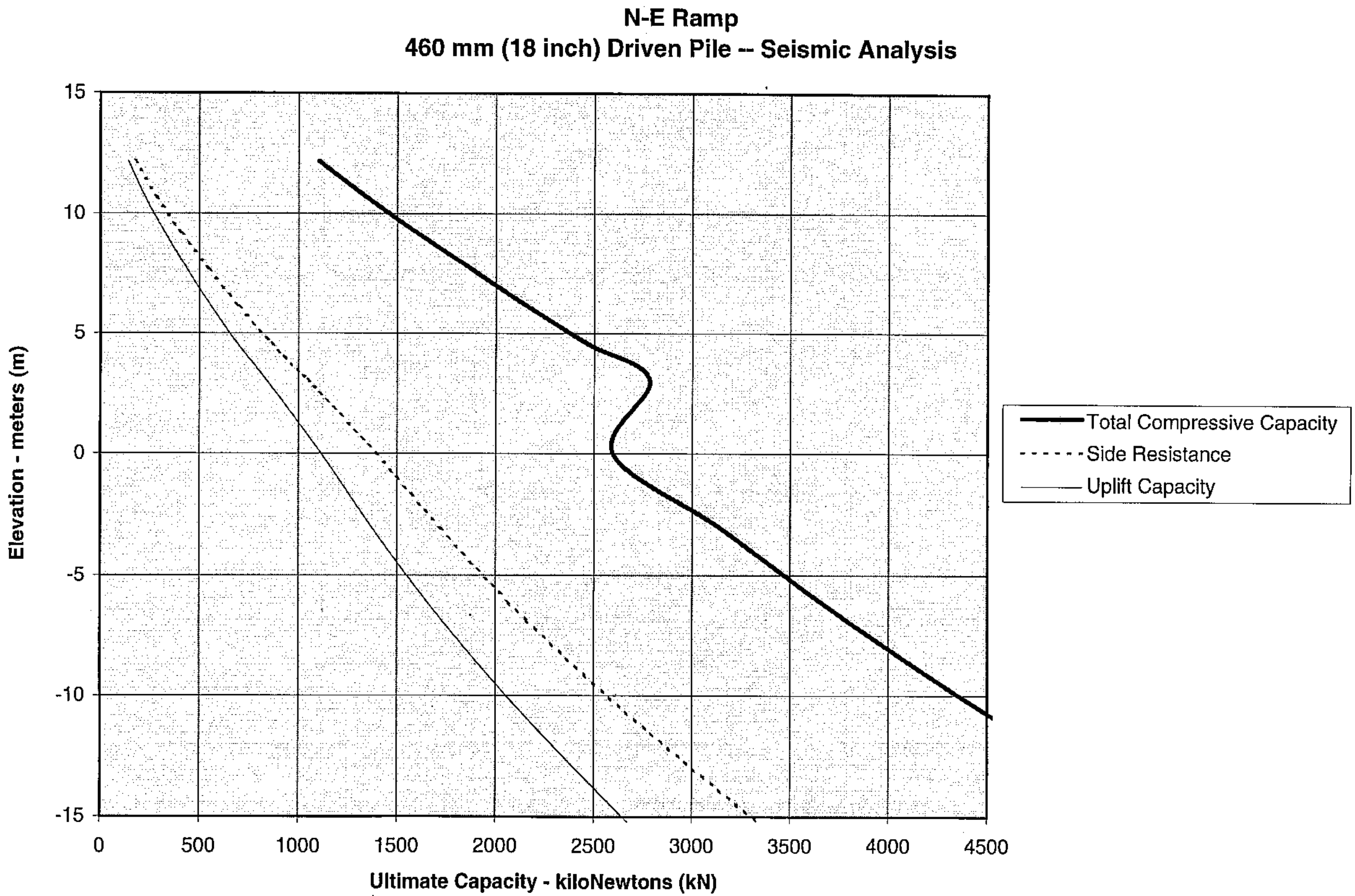


Figure 4-7. Ultimate Pile Capacity for 460 mm (18 inch) Driven, Closed-End Steel Piles at N-E Ramp – Seismic Analysis

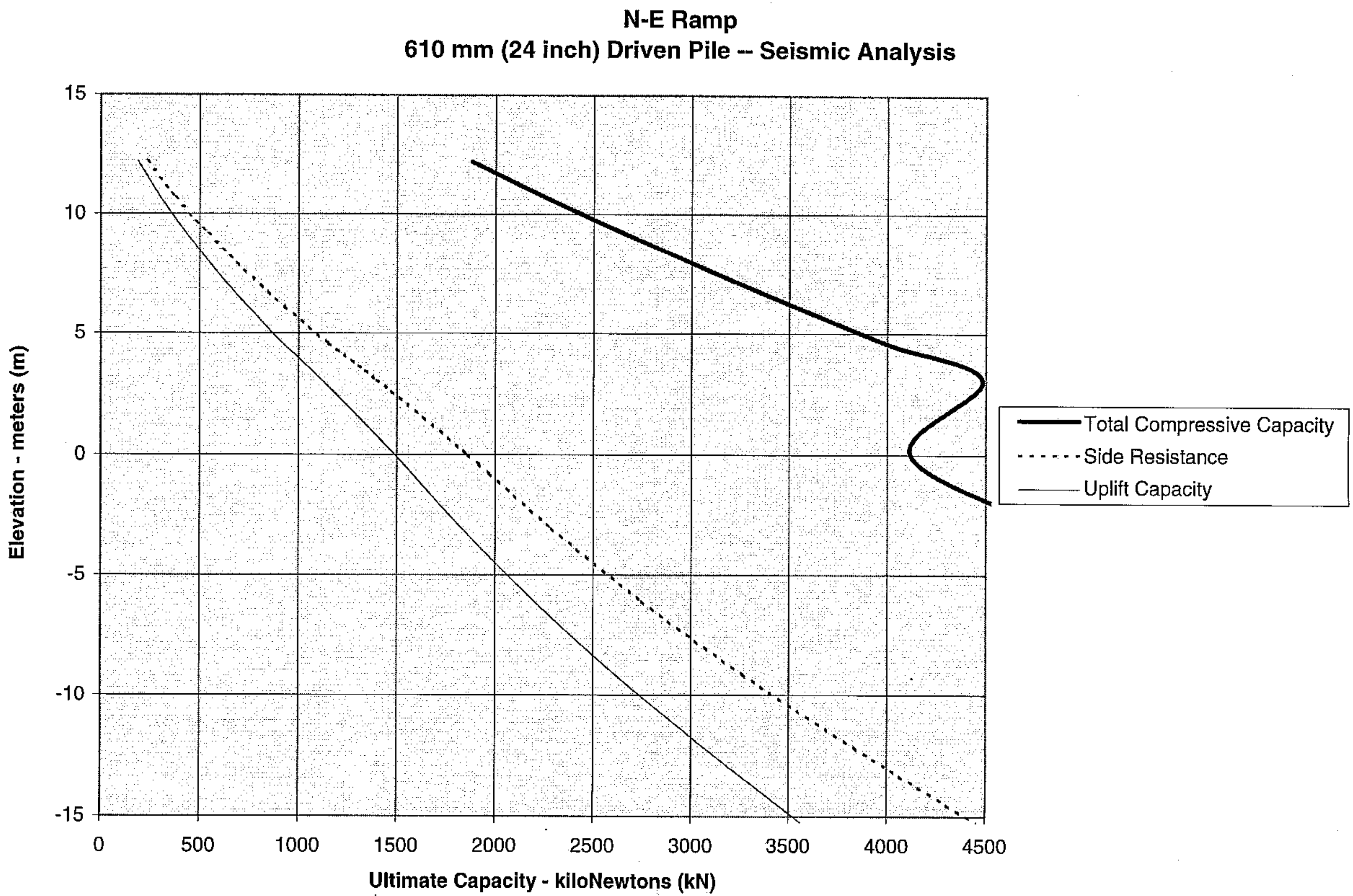


Figure 4-8. Ultimate Pile Capacity for 610 mm (24 inch) Driven, Closed-End Steel Piles at N-E Ramp – Seismic Analysis

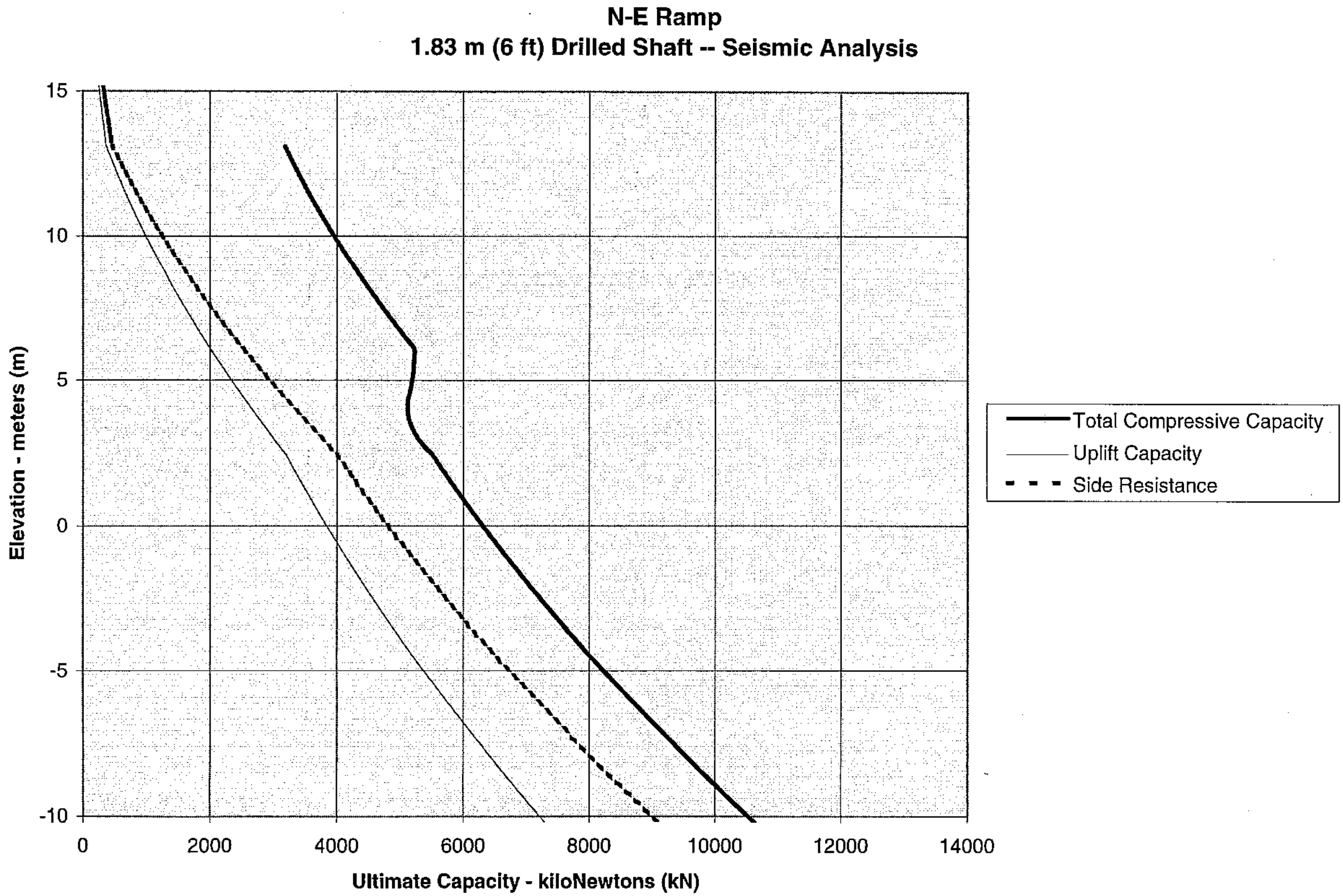


Figure 4-9. Ultimate Drilled Shaft Capacity for 1.83 m (6 ft) at N-E Ramp – Seismic Analysis

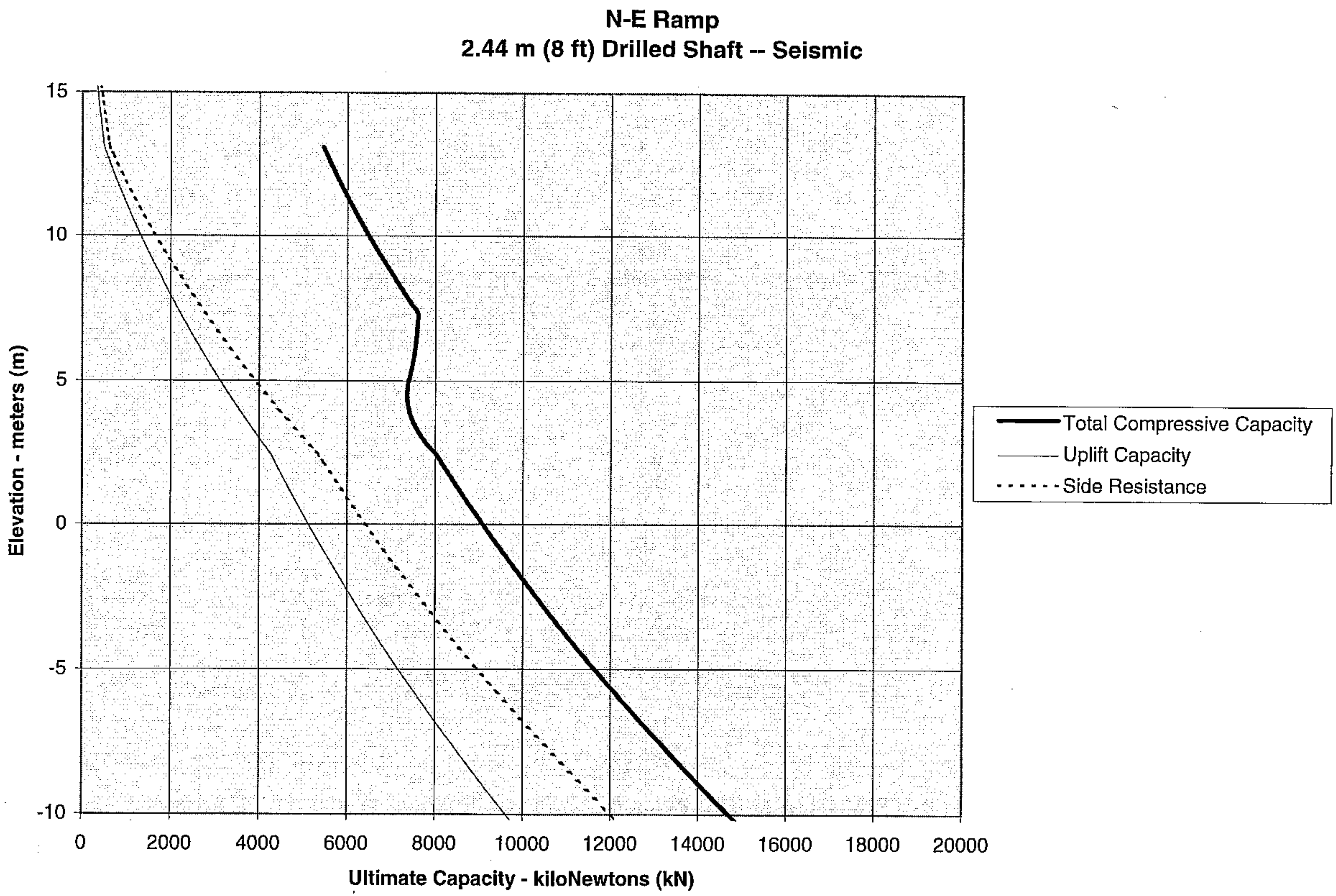


Figure 4-10. Ultimate Drilled Shaft Capacity for 2.44 m (6 ft) at N-E Ramp – Seismic Analysis

1997 SOIL TEST HOLE LOGS

FOR

N-E RAMP

Proj. No.: 116184.G4

SOIL TEST HOLE LOG

Project: SR-167, CS 1765/6, OL-2305 Drilling Contractor: WSDOT

Drilling Method & Equipment: CME 45 Skid Rig, Wash Rotary w/ 100 mm Casing Logger: M. Xue/CivilTech

Depth Below Surface (FT)	Sample: Interval	Sample: Number and Type	Sample: Recovery (FT)	Standard Penetration Test Results 6"-6"-6" (N)	Soil Description: Soil name, uscs group symbol, color, moisture content, relative density or consistency, soil structure, mineralogy	Comments: Depth of casing, drilling rate, drilling fluid loss, tests and instrumentation
Elevation: 28 m (NAVD88)					Location: Sta. 6+53m; Offset 8.5m R of CL	**Test Hole H-1-97**
Start: 12/01/1997					Finish: 12/02/1997	Water Level: ~20 m
						Sheet of 1 of 2
						Start drilling @ 1:45 pm on 12/01/97 with wash rotary
5.0	4.5 - 6.0	S-1	0.6	2-6-6	SANDY SILT, (ML), brown, very moist, stiff, with some gravel and roots (FILL)	Gravels rounded
						Driller notes gravels and cobbes during drilling
10.0	9.5 - 11.0	S-2	0.2	9-10-23	SANDY SILT, (ML), similar to S-1, grading to more gravels and very stiff (FILL)	1.5" gravel stuck in shoe
15.0	14.5 - 16.0	S-3	1.1	7-8-8	SAND., (SP), medium, brown, very moist, medium dense, trace of gravel (FILL)	Driller notes scattered cobbles
20.0	19.5 - 21.0	S-4	0.8	13-7-12	SILTY SAND, (SM), fine to medium, mottled brown to gray, very moist, medium dense, some rounded gravel, (FILL)	Driller notes scattered Cobbles. Slow drilling at 19' through cobbles Driller notes more gravels and cobbles
25.0	24.5 - 26.0	S-5	0.1	8-19-14	SILTY SAND, (SM), similar to S-4, grading to dense (FILL)	Stop drilling @ 4:40 pm Resume drilling @ 8:50 am 12/02/97. Water 5' below ground prior to drilling
30.0	29.5 - 31.0	S-6	0.9	11-25-24	SILTY SANDY GRAVEL, (GM/GP), dark brown, wet, dense, rounded gravel to 1.5" (FILL)	
35.0	34.5 -	S-7	0.6	8-8-19	GRAVELLY SAND,(SP), medium, dark	

NOTES:

1) Test hole locate on south abutment approximately 10' east and 4.6' south of southeast corner of bridge
2) All blowcounts recorded with WSDOT automatic hammer
3) Water encountered in test hole at approximate elevation 20 on 12/03/97.

1 foot = 0.3048 meters
1 inch = 25.4 millimeters

CH2M HILL

Proj. No.: 116184.G4

SOIL TEST HOLE LOG

Project: SR-167, CS 1765/6, OL-2305 Drilling Contractor: WSDOT

Drilling Method & Equipment: CME 45 Skid Rig, Wash Rotary w/ 100 mm Casing Logger: M. Xue/CivilTech

Depth Below Surface (FT)	Sample: Interval	Sample: Number and Type	Sample: Recovery (FT)	Standard Penetration Test Results 6"-6"-6" (N)	Soil Description: Soil name, uscs group symbol, color, moisture content, relative density or consistency, soil structure, mineralogy	Comments: Depth of casing, drilling rate, drilling fluid loss, tests and instrumentation
					Location: Sta. 6+53m; Offset 8.5 R of CL	**Test Hole H-1-97**
Start: 12/01/1997					Finish: 12/02/1997	Water Level:
						Sheet 2 of 2
	36.0				gray, wet, medium dense, some silt (FILL)	
						Driller notes easier drilling at 38.5',
40.0	39.5 - 41.0	S-8	0.9	2-5-6	SANDY SILT, (ML), dark gray, very moist, very fine sand layer (3"), traces organics and gravel, stiff	
					END OF SOIL TEST HOLE AT 41.0 FEET	Stopped drilling at 10:50 am on 12/02/97
45.0						
50.0						
55.0						
60.0						
65.0						
70.0						

NOTES:

Installed piezometer. Total length = 40.5'. Bottom 2' solid casing (1"). 5' of screen with 1/32" slot at 1/4" spacing. Sand pack located in bottome 8.5'. Top 31.5' of piezometer 1" solid pvc casing. Top 30' backfille with bentonite. Locking cap located at the ground surface.

1 foot = 0.3048 meters
1 inch = 25.4 millimeters

CH2M HILL

Proj. No.:116184.G4

SOIL TEST HOLE LOG

Project: SR-167, CS 1765/6, OL-2305 Drilling Contractor: WSDOT

Drilling Method & Equipment: Longyear BK-80 Truck Mounted Rig Logger: R. Devulapally/Terra

Depth Below Surface (FT)	Sample: Interval	Sample: Number and Type	Sample: Recovery (FT)	Standard Penetration Test Results 6"-6"-6" (N)	Soil Description: Soil name, uscs group symbol, color, moisture content, relative density or consistency, soil structure, mineralogy	Comments: Depth of casing, drilling rate, drilling fluid loss, tests and instrumentation
						Start drilling at 11:00 am on 12/01/97. Wash rotary
						Driller notes hard drilling between 2 and 5 feet
5.0	4.0 - 5.5	S-1	0.7	3-4-18	SAND TO SILTY SAND, (SP/SM), fine, gray to black, medium dense, some silt and gravel	
10.0	9.0 - 10.5	S-2	0.4	6-5-5	GRAVEL (GP/GM), gray to black, loose, some sand and silt	
						Driller notes loss of water
15.0	14.0 - 15.5	S-3	0.8	5-5-8	SAND TO SILTY SAND, (SP/SM), fine, black, medium dense,	Top 0.4' siltier
20.0	19.0 - 20.5	S-4	1.4	6-7-9	SAND, (SP), fine, black, medium dense, some silt	
25.0	24.0 - 25.5	S-5	1.5	3-4-4	SILTY CLAY TO CLAYEY SILT, (ML/CL), black, loose, trace of sand	Pocket pen = 0.25 tsf)
30.0	29.0 - 30.5	S-6	0.9	9-9-8	SAND AND GRAVEL, (SP/GP), black, medium dense, rounded gravel	Attempted Shelby tube at 29'. Encountered sand. No recovery
35.0	34.0 -	S-7	1.0	5-6-13	SAND, (SP), black, medium dense, some	

Elevation: 21m (NAVD88) | Location: Sta. 7+21m; Offset 7.0m R of CL | **Test Hole H-2-97**

Start: 12/01/97 | Finish: 12/03/97 | Water Level: Not Measured

Sheet 1of 4

NOTES:

1) Test hole located below N-E Ramp bridge 6' east of bridge and 6' north of pavement next to north approach fill
2) All blowcounts obtained with WSDOT automatic hammer
3) Water encountered in test hole at approximate elevation 19.5m on 12/03/97.

1 foot = 0.3048 meters
1 inch = 25.4 millimeters

CH2M HILL

Proj. No.:116184.G4

SOIL TEST HOLE LOG

Project: SR-167, CS 1765/6, OL-2305 Drilling Contractor: WSDOT

Drilling Method & Equipment: Longyear BK-80 Truck Mounted Rig Logger: R. Devulapally/Terra

Depth Below Surface (FT)	Sample: Interval	Sample: Number and Type	Sample: Recovery (FT)	Standard Penetration Test Results 6"-6"-6" (N)	Soil Description: Soil name, uscs group symbol, color, moisture content, relative density or consistency, soil structure, mineralogy	Comments: Depth of casing, drilling rate, drilling fluid loss, tests and instrumentation
Elevation: 21m (NAVD88)					Location: Sta. 7+21m; Offset 7.0m R of CL	**Test Hole H-2-97**
Start: 12/01/97					Finish: 12/03/97	Water Level: Not Measured
						Sheet 2 of 4
	35.5				gravel	Switched to 3' casing with wash rotary due to gravel
40.0	39.0 - 40.5	S-8	0.6	23-14-26	SAND AND GRAVEL, (SP/GP), black, dense, some silt	100% loss of return water
45.0	44.0 - 45.5	S-9	0.6	6-5-6	SAND AND GRAVEL, (SP/GP), black, medium dense	Stop drilling at 4:00 pm Resume drilling at 8:00 am on 12/02/97. Augered hole with 4 1/4" HSA. Water at 27'
50.0	49.0 - 50.5	S-10	1.5	5-9-24	SILTY SAND TO SANDY GRAVEL, (SP/SM/GP), black, dense	Void at 51-52'
55.0	54.0 - 55.5	S-11	0.6	14-12-16	SANDY GRAVEL, (GP), black, medium dense, some silt	Heave in augers (~3') at about 54'. Washing out sand Seated S-11 by 6"
60.0	59.0 - 60.5	S-12	1.1	5-13-12	SAND, (SP), black, medium dense, with gravel and trace of silt	Seated S-12 by 6"
65.0	64.0 - 65.5	S-13	1.2	5-5-6	SANDY SILT, (ML), black, medium dense, trace of wood pieces and organics.	Heave in augers, Washed out.
70.0	69.0 -	S-14	1.5	2-3-12	SILT (ML), black, medium dense, with	Switched to 3" casing with wash rotary

NOTES:

1 foot = 0.3048 meters
1 inch = 25.4 millimeters

CH2M HILL

Proj. No.:116184.G4

SOIL TEST HOLE LOG

Project: SR-167, CS 1765/6, OL-2305 Drilling Contractor: WSDOT

Drilling Method & Equipment: Longyear BK-80 Truck Mounted Rig Logger: R. Devulapally/Terra

Depth Below Surface (FT)	Sample: Interval	Sample: Number and Type	Sample: Recovery (FT)	Standard Penetration Test Results 6"-6"-6" (N)	Soil Description: Soil name, uscs group symbol, color, moisture content, relative density or consistency, soil structure, mineralogy	Comments: Depth of casing, drilling rate, drilling fluid loss, tests and instrumentation
Elevation: 21m (NAVD88)					Location: Sta. 7+21m; Offset 7.0m R of CL	**Test Hole H-2-97**
Start: 12/01/97					Finish: 12/03/97	Water Level: Not Measured
						Sheet 3 of 4
	70.5				some clay, wood fragments	Pocket pen = 0.75 tsf, sand at 70' to 70.5'
75.0	74.0 - 75.5	S-15	0.8	3-5-3	SANDY SILT, (ML), black, loose, trace of gravel, some wood fragments	
80.0	79.0 - 80.5	S-16	1.2	2-5-15	SANDY SILT, (ML), black, medium dense, some gravel, some wood fragments fragments	
						Stop drilling at 3:15 pm Water at 5' below ground surface at start of drilling on 12/03/97
85.0	84.0 - 85.5	S-17	1.5	2-1-5	SILTY SAND, (SM), black, loose, some gravel	
90.0	89.0 - 90.5	S-18	1.5	3-5-5	SILTY SAND, (SM), black, loose, some gravel	
95.0	94.0 - 95.5	S-19	1.5	1-3-3	SILTY SAND, (SM), black, loose, some gravel	
100.0	99.0 - 100.5	S-20	0.5	4-4-4	SILTY SAND, (SM), black, loose, trace of gravel	
105.0	104.0 -	S-21	0.5	4-5-3	SILTY SAND, (SM), black, loose, with	

NOTES:

1 foot = 0.3048 meters
1 inch = 25.4 millimeters

CH2M HILL

Proj. No.:116184.G4

SOIL TEST HOLE LOG

Project: SR-167, CS 1765/6, OL-2305 Drilling Contractor: WSDOT

Drilling Method & Equipment: Longyear BK-80 Truck Mounted Rig Logger: R. Devulapally/Terra

Elevation: 21m (NAVD88) | Location: Sta. 7+21m; Offset 7.0m R of CL | **Test Hole H-2-97**

Start: 12/01/97 | Finish: 12/03/97 | Water Level: Not Measured

Depth Below Surface (FT)	Sample: Interval	Sample: Number and Type	Sample: Recovery (FT)	Standard Penetration Test Results 6"-6"-6" (N)	Soil Description: Soil name, uscs group symbol, color, moisture content, relative density or consistency, soil structure, mineralogy	Comments: Depth of casing, drilling rate, drilling fluid loss, tests and instrumentation
	105.5				trace of gravel	
110.0	109.0 - 110.5	S-22	0.2	3-4-7	SILTY SAND, (SM), black, medium dense, some gravel	
115.0	114.0 - 115.5	S-23	0.2	11-10-8	SILTY SAND, (SM), black, medium dense, with some gravel	
120.0	119.0 - 120.5	S-24	1.5	8-13-7	SILTY SAND, (SM), black, medium dense, some gravel	
					END OF SOIL TEST HOLE AT 120.5 FEET	Stopped drilling at 3:00 pm on 12/03/97
125.0						
130.0						
135.0						
140.0						

NOTES:

1 foot = 0.3048 meters
1 inch = 25.4 millimeters

CH2M HILL

Proj. No.: 116184.G4

SOIL TEST HOLE LOG

Project: SR167, CS 1765/6, OL-2305 Drilling Contractor: WSDOT

Drilling Method & Equipment: CME 45 Skid Rig, Wash Rotary w/ 100 mm Casing Logger: M. Xue/CivilTech

Depth Below Surface (FT)	Sample: Interval	Sample: Number and Type	Sample: Recovery (FT)	Standard Penetration Test Results 6"-6"-6" (N)	Soil Description: Soil name, uscs group symbol, color, moisture content, relative density or consistency, soil structure, mineralogy	Comments: Depth of casing, drilling rate, drilling fluid loss, tests and instrumentation
Elevation: 28m (NAVD88)					Location: Sta. 7+47m, Offset 6.4m R of CL	**Test Hole H-3-97**
Start: 12/02/97					Finish: 12/03/97	Water Level: Not Measured
						Sheet 1 of 2
						Started drilling at 3:23 pm on 12/02/97 Driller notes trace of cobbles
5.0	4.6 - 6.0	S-1	0.9	5-7-7	SILTY SAND, (SP/SM), fine to medium, dark brown, moist, medium dense, some silt and subrounded gravel (FILL)	Slow drilling due to cobbles
10.0	9.5 - 11.0	S-2	1.0	6-9-9	SAND TO SILTY SAND, (SP/SM), medium, dark brown, moist, medium dense, some silt and gravel (FILL)	Stopped at 4:20 pm Resumed drilling at 8:50 am on 12/3/1997
15.0	14.5 - 16.0	S-3	0.0	10-10-9	NO RECOVERY, driller noted the same material as above (FILL)	
20.0	19.5 - 21.5	S-4	0.5	9-10-11-10	SANDY SILTY GRAVEL, (GP/GM), fine to coarse gravel, brown, very moist, medium dense, subrounded (FILL)	Sampler driven 24" for recovery.
25.0	24.5 - 26.5	S-5	1.1	16-30-21-18	SILTY SAND AND GRAVEL, (SM/GM), fine to coarse sand and gravel, mottled brown gray and orange, moist to wet, dense to very dense, subrounded to angular gravel (FILL)	Sampler driven 24" for recovery
30.0	29.5 31.0	S-6	0.6	50/2" 183/121/96)	SANDY GRAVEL, (GP), fine to coarse, angular, gray, very moist to wet, very dense some silt (FILL)	High blowcount was in the first 6". Sampler driven 18" to retrieve sample.
35.0	34.5 -	S-7	0.4	8-7-10	SANDY GRAVEL, (GP), coarse sand, fine	Changed to HQ wireline

NOTES:

1) Test hole locate on north abutment approximately 7' east and 5' north of northeast corner of bridge.
2) All blowcounts obtained with WSDOT automatic hammer
3) Water not measured. Water in test hole from rotary wash drilling method.

1 foot = 0.3048 meters
1 inch = 25.4 millimeters

CH2M HILL

Proj. No.: 116184.G4

SOIL TEST HOLE LOG

Project: SR167, CS 1765/6, OL-2305 Drilling Contractor: WSDOT

Drilling Method & Equipment: CME 45 Skid Rig, Wash Rotary w/ 100 mm Casing Logger: M. Xue/CivilTech

Depth Below Surface (FT)	Sample: Interval	Sample: Number and Type	Sample: Recovery (FT)	Standard Penetration Test Results 6"-6"-6" (N)	Soil Description: Soil name, uscs group symbol, color, moisture content, relative density or consistency, soil structure, mineralogy	Comments: Depth of casing, drilling rate, drilling fluid loss, tests and instrumentation
Elevation: 28m (NAVD88)					Location: Sta. 7+47m, Offset 6.4m R of CL	Test Hole H-3-97
Start: 12/02/97					Finish: 12/03/97	Water Level: Not Measured
						Sheet 2 of 2
	35.5				to coarse gravel, dark gray, wet, medium dense, subrounded to angular gravel, traces of silt (FILL)	Driller notes cobbles at 37'
40.0	39.0 - 40.5	S-8	1.1	6-8-12	SILT, (ML), fine, dark gray, very moist, stiff, with fine sand and trace of gravel	3" thick sand layer at bottom of sampler
					END OF SOIL TEST HOLE AT 40.5 FEET	Stopped drilling at 2:40 pm on 12/03/97
45.0						
50.0						
55.0						
60.0						
65.0						
75.0						

NOTES:

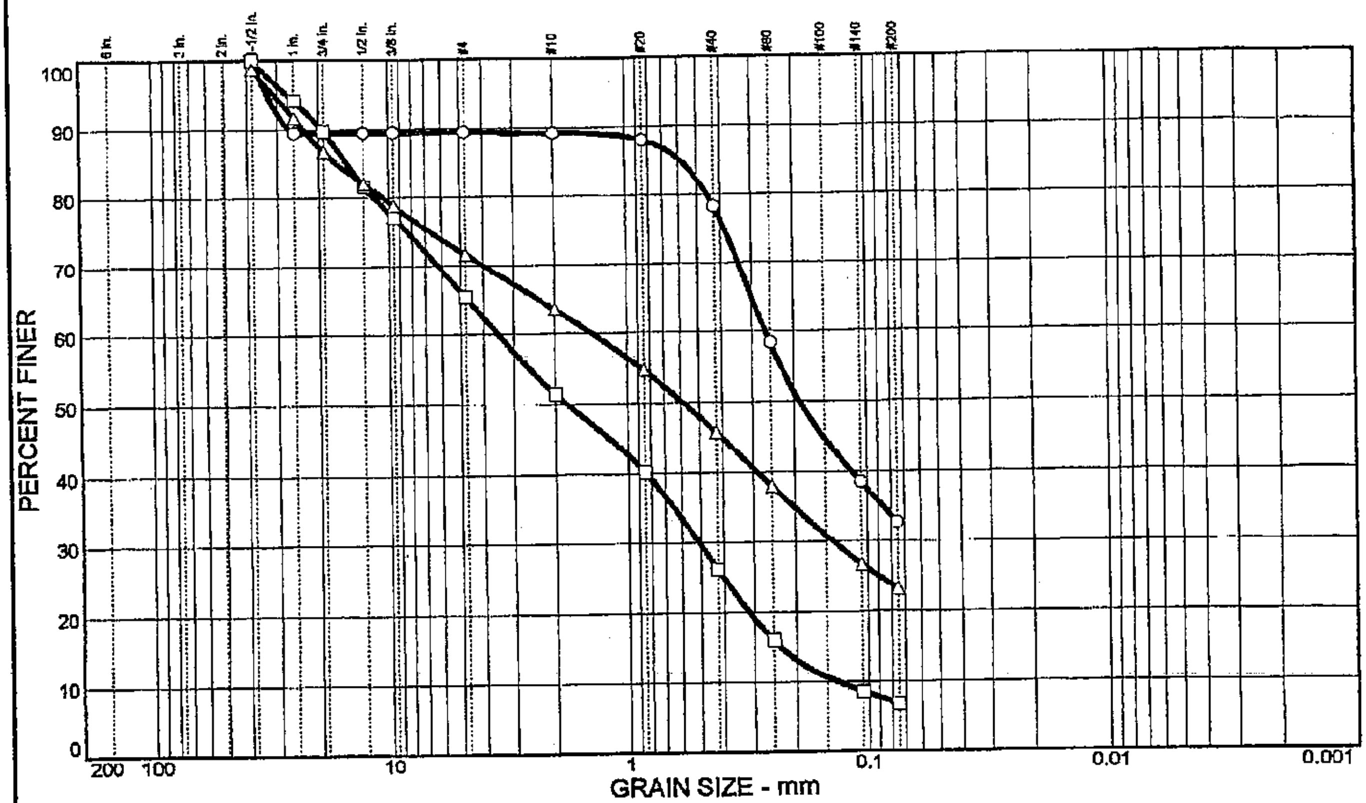
1 foot = 0.3048 meters
1 inch = 25.4 millimeters

1997 LABORATORY TEST DATA

FOR

N-E RAMP

PARTICLE SIZE DISTRIBUTION TEST REPORT



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○		10.6	56.7			SM			
□		34.6	58.6			SP-SM			
△			48.4			SM			

SIEVE inches size	PERCENT FINER ○	□	△
1.5	100.0	100.0	98.8
1	89.4	94.2	91.7
.75	89.4	89.7	86.7
.5	89.4	81.7	82.1
.375	89.4	77.1	78.8
GRAIN SIZE			
D_{60}	0.262	3.48	1.40
D_{30}		0.506	0.140
D_{10}		0.136	
COEFFICIENTS			
C_c		0.54	
C_u		25.59	

SIEVE number size	PERCENT FINER ○	□	△
#4	89.4	65.4	71.6
#10	89.2	51.4	63.5
#20	88.0	40.2	54.6
#40	78.2	26.2	45.7
#60	58.3	16.0	38.0
#140	38.5	8.6	26.7
#200	32.7	6.8	23.2

SOIL DESCRIPTION

○ Silty sand (SM)

□ Poorly graded sand with silt and gravel (SP-SM)

△ Silty sand with gravel (SM)

REMARKS:

○

□

△

○ Source: H-2 Sample No.: 3-S Elev./Depth: 15-16 ft

□ Source: H-2 Sample No.: 10-S Elev./Depth: 50-51 ft

△ Source: H-2 Sample No.: 17-S Elev./Depth: 85-86 ft

SOIL TECHNOLOGY, INC.

Client: CH2M Hill

Project: WSDOT On Call Geotech Services 116184.G4.03

Project No.: J-1118 Plate 1

CH2M Hill WSDOT On Call Geotech Services
116184.G4.03

Percent Passing U.S. Sieve #200
Table 1

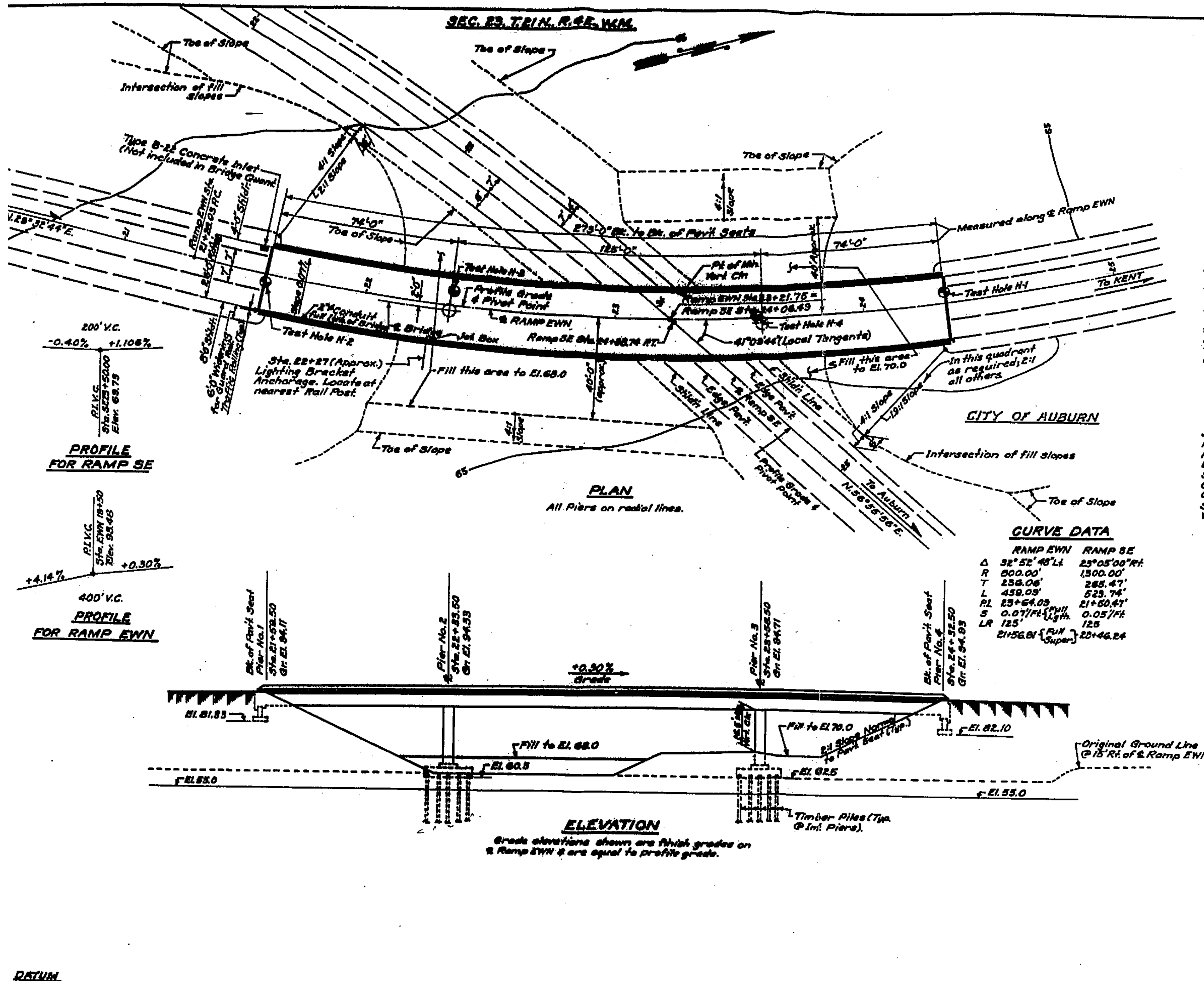
Boring Number	Sample Number	Depth (ft)	Percent Passing 75 micron
H-1	SPT-3	14.5-16.0	26
H-2	1-S	5.0-6.0	9
H-2	4-S	20.0-21.0	34
H-2	5-S	25.0-26.0	68
H-3	SPT-2	9.5-11.0	24
H-3	SPT-8	39.0-40.0	56

EXISTING DATA

FOR

N-E RAMP

FED. ROAD DIV. NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
8	WASH.	U-021-1(3)	120	170



GENERAL NOTES

All material and work shall be in accordance with the requirements of the State of Washington, Department of Highways, Standard Specifications for Road and Bridge Construction, dated 1969.

Footing elevations are subject to change depending upon foundation material encountered. Reinforcing steel for the footings, columns & walls shall not be cut until final footing elevations have been determined in the field.

The concrete in the footings of all piers and the walls of Piers No. 1 and 4 shall be Class B mix. All other cast in place concrete shall be Class AX mix.

Falsework shall not be released in any span until all concrete, except that in the rail base, has been in place the required length of time and has developed sufficient strength as outlined in the specifications. Falsework shall be carefully released to prevent impact or undue stresses in the structure. The railbase shall not be poured until the falsework has been released.

The maximum design soil pressure per square foot is three (3) tons for Piers No. 1 & 4.

Each pile shall be driven to a depth sufficient to develop a load bearing capacity of forty (40) tons.

Unless otherwise shown on the plans, concrete cover measured from the face of the concrete to the face of any reinforcement bar shall be 2" at the top of the roadway slab, 1" at the bottom of the Rdwy. Slab & top & bottom of bottom slabs, 2½" at the bottom of footings and 1½" at all other locations.

APPROXIMATE QUANTITIES

Structure Excavation Class A.	520	Cu. Yds.
Furnishing and Driving Timber Test Piles.	2	Only
Furnishing Timber Piling (Creosote Treated).	2,100	Lin. Ft.
Driving Timber Piles (Creosote Treated.)	56	Only
Steel Reinforcing Bars.	28,500	Lbs.
Concrete Class B.	120	Cu. Yds.
Concrete Class AX.	30	Cu. Yds.
Superstructure, Ramp EWN O-Xing		Lump Sum
Water Reducing Additive.	Est. 375	Dollars

LOADING: HS-20 OR TWO 24K AXLES @ 4' CTR'S.

SR 167 MP 13.77 TO MP 14.73

15TH ST. S.W. TO W. MAIN ST. IN AUBURN

KING COUNTY

RAMP EWN OVERCROSSING

LAYOUT

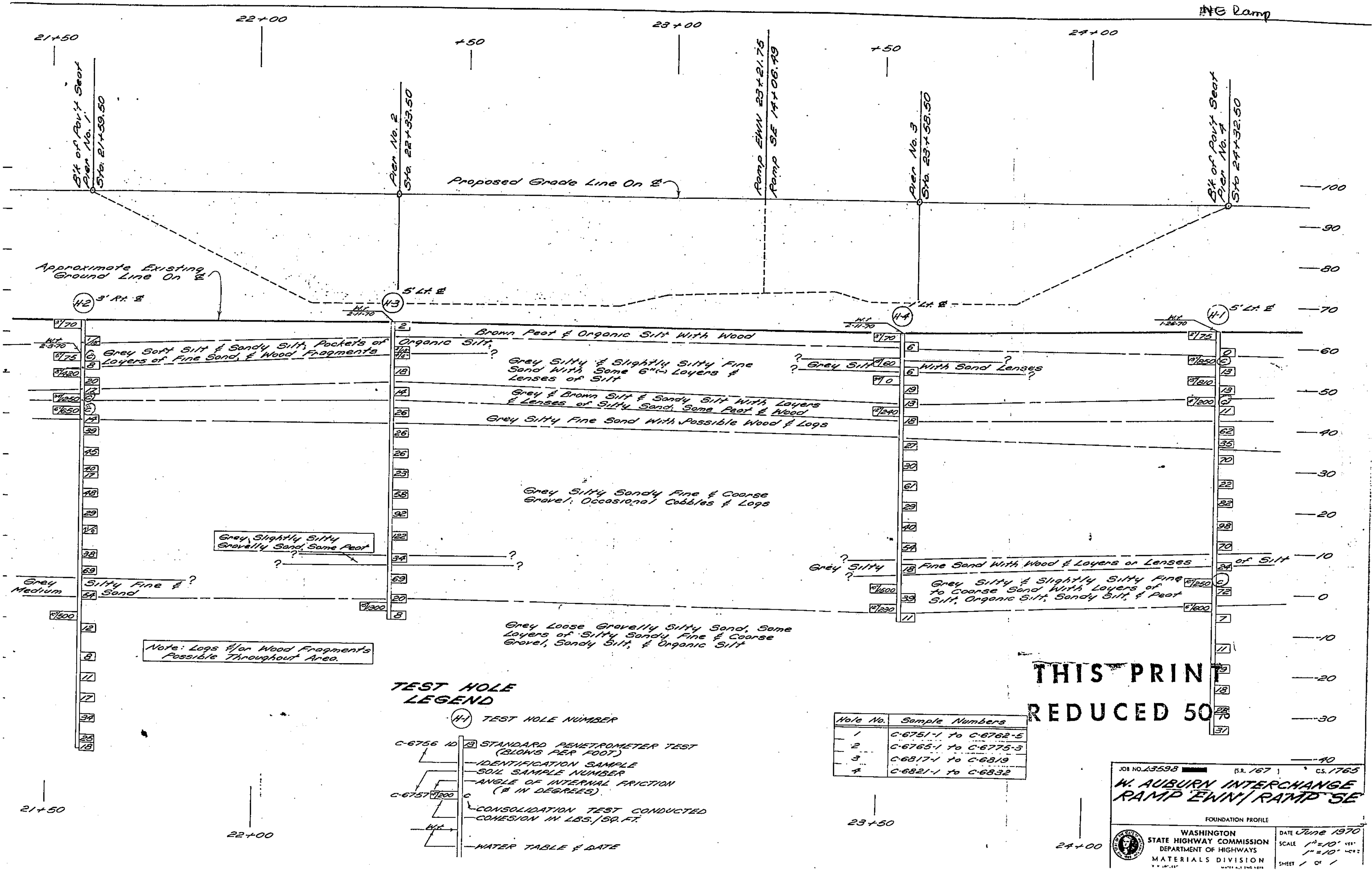
WASHINGTON STATE HIGHWAY COMMISSION
DEPARTMENT OF HIGHWAYS
OLYMPIA, WASHINGTON

October 4, 1971

9236

SHEET 120 OF 170 SHEETS

SR 167 SR 167/51



Hole No.	Sample Numbers
1	C-6751-1 to C-6762-5
2	C-6765-1 to C-6775-5
3	C-6817-1 to C-6819
4	C-6821-1 to C-6832

JOB NO. L3598 [redacted] (S.R. 167) C.S. 1765

W. AUBURN INTERCHANGE
RAMP "EWN" / RAMP "SE"

FOUNDATION PROFILE

WASHINGTON STATE HIGHWAY COMMISSION
DEPARTMENT OF HIGHWAYS
MATERIALS DIVISION

DATE June 1970
SCALE 1" = 10' VERT. 1" = 10' HORIZ.
SHEET 1 OF 1

NE-Ramp

. F. 26.66 (Rev. 5-67)

WASHINGTON
STATE HIGHWAY COMMISSION
DEPARTMENT OF HIGHWAYS

Original to Materials Engineer
Copy to Bridge Engineer
Copy to District Engineer
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LOG OF TEST BORING

S.H. S.R. 167 Section West Auburn Interchange Job No. L-3598
Hole No. H-1 Sub Section Ramp A over Crossing (A/E) Cont. Sec. 176501
Station 24+31 Offset 5' Left of ℄ Ground El. 64.0'
Type of Boring Chop & Drive Casing 96'0" X 3" W.T. El. Note at end
Inspector LeRoy R. Sampson Date January 26, 1970 Sheet 1 of 5

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
			A C	Ground surface submerged beneath 2'6" of swamp water.
			C U-1	PEAT - very soft, saturated, reddish brown, roots
				and wood fragments scattered through.
				logs or wood fragments possible through the
5	4/24"		1/18" Std. Pen.	depth of this boring, (A log noted at 31').
			2 3	SILTY, FINE, SAND WITH SILT LENSES - Slightly
			A B	compact, dark gray sand, 6" ± lenses of
			D E U-3	soft, gray, moist silt.
			6 Std.	
10	13		6 7 Pen. 4	
			8	
			A B U-5	
			3 Std. 6 Pen.	
15	13		7 6 4	
			A C B U-7	rotted wood fragments 16' to 18'.
			D	
			5 Std. 6 Pen.	
20	11		5 8 11	

Hole No. H-1 Sub Section Ramp A Over Crossing (A/E) Sheet 2 of 5

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
				SANDY GRAVEL - Dense, gray, fine to coarse
				sand and fine to coarse gravel
			23 Std.	
	62		30 Pen.	
25			32 9 30	A trace of silt
				Logs and wood scattered through
			20 Std.	
	35		18 Pen. 17 10	
			12	
				SANDY GRAVEL - compact, gray, fine to coarse
30				sand, fine to coarse gravel and wood.
			Std.	A log of about 12" diameter at 31'.
			70 Pen. 30 11	A trace of silt
			$7\frac{1}{2}$"	
35				
			20 Std.	
	22		11 Pen. 11 12	
			6	
				SANDY GRAVEL - dense, gray, moist
40				fine to coarse sand and fine to
				coarse gravel, a trace of silt.
			16 Std.	
	82		22 Pen. 60 13	
45				

Hole No. H-1 Sub Section Ramp A over Crossing (A/E) Sheet 3 of 5

H	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	98		60 44 54 Std. Pen. 14	
50				
	70		40 45 25 17 Std. Pen. 15	
				SILTY, FINE SAND WITH SILT LENSES -
55				compact, moist, dark gray, fine sand
				with rotted wood scattered through.
	24		5 8 16 25 Std. Pen. 16	scattered 6" + lenses of gray, moist silt.
				FINE TO COARSE SAND - dense, dark gray,
				a trace of silt, (this stratum heaves)
60				(no circulation water loss)
			A B U-17	
	72		24 42 30 29 Std. Pen. 18	
65				ORGANIC SILT - very soft, moist, black,
				possibly logs of wood fragments.
			A B C D E U-19	SILTY, SANDY, GRAVEL - loose, gray, moist, [illegible]
				fine to coarse sand and fine to
			4 Std.	coarse gravel.
70	7		4 3 Pen. 20	

Hole No. H-1 Sub Section Ramp A Over Crossing (A/E) Sheet 4 of 5

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
			10 (20 - over	drive on coarse gravel or cobble
75				
	11		6 6 5 6 Std. Pen. 21	
80				
	19		10 11 8 9 Std. Pen. 22	
85				
	18		10 9 9 11 Std. Pen. 23	
90				
	28		18 15 13 11 Std. Pen. 24	
95				

Hole No. H-1 Sub Sectic. Ramp A Over Crossing (A/E) Sheet 5 of 5

H	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	31		10 13 18 15 Std. Pen. 25	test boring stopped at 98'0"
				surface water $2\frac{1}{2}$' over ground surface
				water level with casing varies from
				-7' to -12' below ground surface.

NE - Ramp

F. 26.66 (Rev. 5-67)

WASHINGTON
STATE HIGHWAY COMMISSION
DEPARTMENT OF HIGHWAYS

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LOG OF TEST BORING

S.H. ______ S.R. 167 Section West Auburn Interchange Pier #3 Job No. L-3598

Hole No. H-4 Sub Section Ramp A Over Crossing (A/E) Cont. Sec. 176501

Station A 23+55 Offset 1' Left of Line A Contour Ground El. 62.5'

Type of Boring Chop and Drive Casing 66'0" X 3" W.T. El. 62.5

Inspector LeRoy R. Sampson Date February 19, 1970 Sheet 1 of 3

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
			A B U-1 E F	TOP SOIL - Peat and Organic silt - very soft, brown, saturated small wood fragments scattered through. (2' of water over ground surface)
	6		5 4 2 2 Std. Pen. 2	Very fine sand - very loose, gray, wet
5				
			A B U-3	SILT WITH SAND LENSES - very soft, gray, 3"± lenses of dark gray, fine sand.
10	6		1 2 4 9 Std. Pen. 4	FINE SAND - Slightly compact, dark gray, damp, $\frac{1}{4}$"± lenses of gray silt.
			B C U-5	Logs or wood fragments possible through the depth of this boring.
15	19		4 8 11 11 Std. Pen. 6	
	13		6 6 7 5 Std. Pen. 7	SILT - Soft, gray, moist, small pieces of brown peat scattered through.
20			A B C D U-8	

Hole. No. H-4 Sub Section Ramp A Over Crossing (A/E) Sheet 2 of 3

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	18		4 7 11 11 Std. Pen. 9	FINE SAND - Slightly compact to compact dark gray, damp.
25				SANDY GRAVEL - Compact to dense, gray, fine to coarse sand and fine to coarse gravel, a trace of silt.
	27		23 13 14 24 Std. Pen. 10	
30				
	30		22 16 14 16 Std. Pen. 11	
35				
	61		27 30 31 Std. Pen. 12	
40				
	29		19 12 17 21 Std. Pen. 13	
45				

Hole No. H-4 Sub Section Ramp A Over Crossing (A/E) Sheet 3 of 3

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	40		20, 23, 17, 33 Std. Pen. 14	
50				
	54		24, 24, 30, 29 Std. Pen. 15	
55				FINE SANDY SILT - Slightly compact, gray, damp
	18		8, 9, 9, 17 Std. Pen. 16	
				SAND AND SILT - Layered - gray, moist
60				SILT with wood and peat scattered through and alternate 1' + layers
			B C D E U-17	of dark gray fine sand. Slightly compact silt, compact to dense sand.
~~35~~ 65	39		4, 11, 28, 32 Std. Pen. 18	Logs possible in this stratum.
			A B D E U-19	SILTY, SANDY GRAVEL - Loose to slightly compact, gray, moist, fine to coarse
70	11		3, 4, 7, 5 Std. Pen. 20	sand and fine to coarse gravel. test boring stopped at 70'0"

H.E. Ramp

F. 26.66 (Rev. 5-67)

WASHINGTON
STATE HIGHWAY COMMISSION
DEPARTMENT OF HIGHWAYS

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LOG OF TEST BORING

S.H. ____ S.R. 167 Section West Auburn Interchange Job No. L-3598
Hole No. H-3 Sub Section Ramp A Overcrossing (A/E) Cont. Sec. 176501
Station A 22+33 Pier No. 2 Offset 5' Lt. of A Line Contour Ground El. 63.5'
Type of Boring Chop and Drive Casing 3" x 68'6" W.T. El. 63.5
Inspector LeRoy R. Sampson, E. Duvall Date February 11, 1970 Sheet 1 of 4

Depth	Blows per ft.	Profile	Sample Tube Nos.	Description of Material
	2		1, 1, 1, 1 Std. Pen. 1	1'0" of swamp water over ground surface.
				PEAT - Very soft, saturated, brown, small pieces of wood scattered through.
				VERY FINE SANDY SILT - Very soft, moist, gray, pieces of wood and peat scattered through
	2/18"		1/12", 1, 4 Std. Pen. 2	
				FINE SAND - Slightly compact, dark gray.
				Logs or wood fragments possible through the depth of this boring.
	18		6, 7, 11, 13 Std. Pen. 3	
				SILT WITH SAND LENSES AND WOOD - slightly compact, brownish gray, moist silt with peat, wood fragments and 6" ± lenses of dark gray, fine sand scattered through.
	14		3, 6, 8, 7 Std. Pen. 4	
				FINE SAND - Compact, dark gray.

Hole No. H-3 Sub Section Ramp A Overcrossing (A/E) Sheet 2 of 4

TH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	26		16 14 12 9 Std. Pen. 5	
				SANDY GRAVEL - Compact, dark gray, fine to coarse sand and fine to coarse gravel, probably water bearing.
	26		27 15 11 12 Std. Pen. 6	
				A trace of silt beginning at about 30'.
	26		20 14 12 15 Std. Pen. 7	
	23		17 12 11 14 Std. Pen. 8	SANDY GRAVEL - Dense, gray, fine to coarse sand and fine to coarse gravel with cobbles beginning at about 47', a trace of silt.
	58		30 32 26 25 Std. Pen. 9	

Hole No. H-3 Sub Section Ramp A Overcrossing (A/E) Sheet 3 of 4

BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
			(Dynamite used at 48'6")
92		48 42 50 Std. Pen. 10	Cobbles beginning at 47'
122		50 80 42 35 Std. Pen. 11	
34		40 21 13 23 Std. Pen. 12	SAND - Gravelly, gray, slightly silty, trace brown peat
			SANDY GRAVEL - Dense, gray fine to coarse sand, fine to coarse gravel scattered cobbles, trace silt
69		41 36 33 33 Std. Pen. 13	
			1st 6" D#14 past cobble
20		121 31 10 10 Std. Pen. 14	Alternating thin layers, silt fine sandy, sand, sandy gravel & silty sandy gravel loose to slightly compact, gray, damp to wet
		B C D E U-15	

Hole No. H-3 Sub Section Ramp A Overcrossing (A/E) Sheet 4 of 4

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
			3	
	8		4 Std.	
			4 Pen.	
			6 16	
				STOPPED TEST BORING AT 72'6"

-3598

TIME - SETTLEMENT COMPUTATIONS

SH# ______ SECTION RAMP "A" O'-XING A/E ______ STA ______ HOLE# ______

t	$\sqrt{t}$	500	1000	2000	4000	8000
/4	0.5			49	56	
1/2	0.707			52	60	
1	1			58	65	
2 1/4	1.5			64	73	
4	2			71	77	
9	3			80	83	
6	4			85	86	
5	5			86	89	
36	6			89	91	
9	7			89	92	

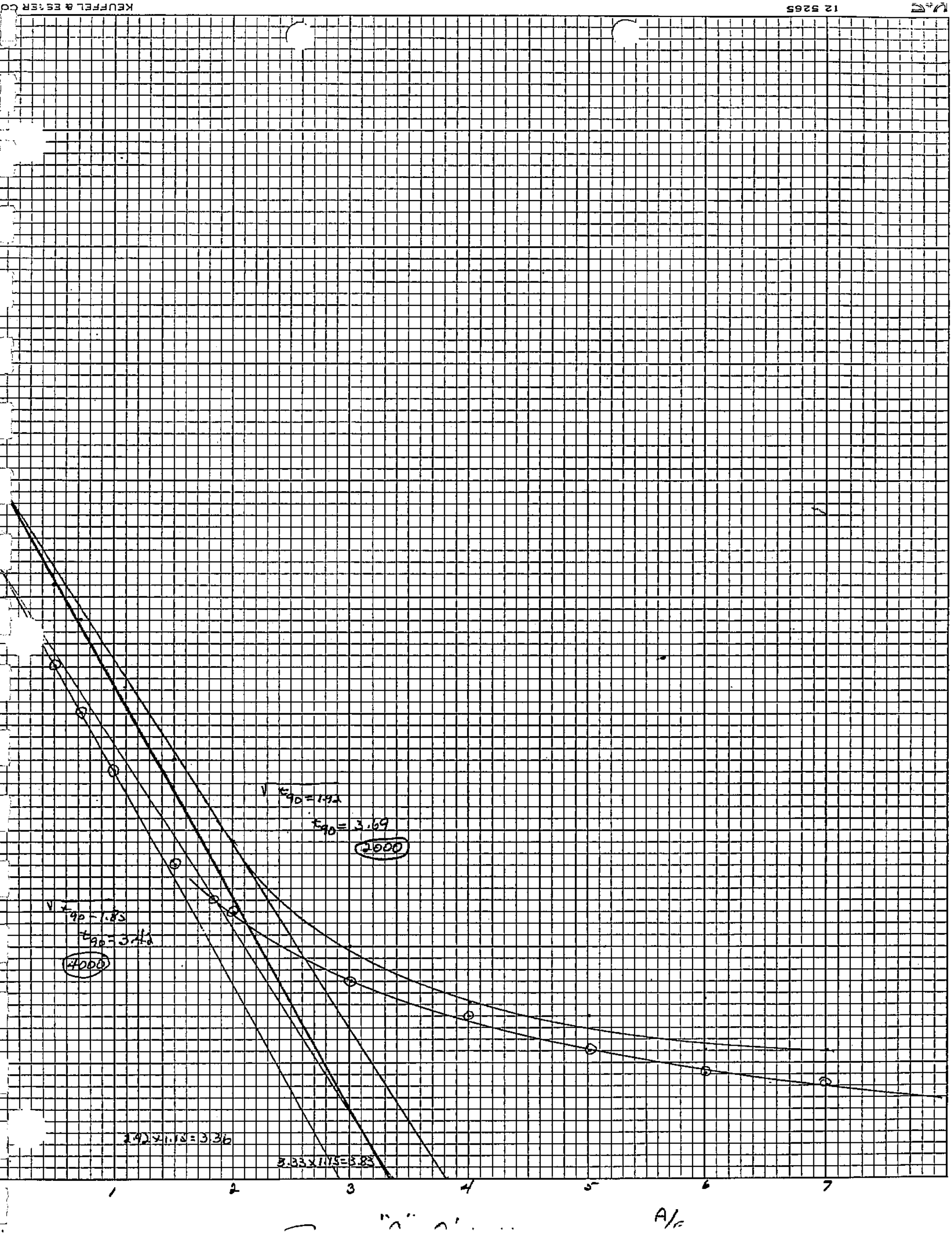
%S	S	N_1	$t=N_1\left(\frac{.405d^2}{c}\right)$	N	$t=N\left(\frac{.405d^2}{c}\right)$
10		0.02	yrs		
20		0.08			
30		0.17			
40		0.31			
50		0.49			
60		0.71			
70		1.00	0.0474	0.1310	
80		1.40	.0663	0.183	
90		2.09	0.0990	0.274	
100		∞	∞	∞	∞

MPLE NO. C-6758-3 / 6757-3 / 6767-3 LAYER ①

$$\frac{0.405\,d^2}{c} = \frac{.405\left(\frac{6}{2}\right)^2}{77} = .0474$$

$$= \frac{.405\left(\frac{10}{2}\right)^2}{77} = 0.1310$$

LOAD	h_i (in)	Δh	h_f	$\frac{h_i+h_f}{4}$	$\left(\frac{h_i+h_f}{4}\right)^2$	t_{90} (min)	$c = \frac{3095}{t_{90}}\left(\frac{h_i+h_f}{4}\right)^2$ (ft²/yr)	
	0.6250	.0214	.6036	.3072	.09437			
00	.6036	.0058	.5978	.3004	.09024			c= @
00	.5978	.0078	.5900	.2970	.08821	3.69	74.0	c= @
000	.5900	.0103	.5797	.2924	.08550	3.42	77.4	c= @
00	.5797	.0099	.5698	.2874	.08260			c= @



√t90 = 1.92
t90 = 3.69
2000
√t90 = 1.85
t90 = 3.42
4000
1.92 x 1.15 = 3.36
3.33 x 1.15 = 3.83
1
2
3
4
5
6
7
A/c

Date 3-6-70 — 3-13-70 Sample No. C-6753-3

No. L-3598 Section RAMP A/E

Dep 7'0" 7'4" Station A24+31 5' Lt Hole No. H-1

Material GREY SILTY SAND

Gross Weight	Tare	Net Weight	Can Number	Wet Weight	Dry Weight	Weight H_2O	% H_2O	Wet Density	Dry Density
498	154	344	54	70.84	52.70	18.14	34.4	113.7	84.6
		Consol	48	54.03	40.31	13.72	34.0		

Sample Height = 4.0

= Wt. Dry Soil / K

= hv_o / h_s

= 47.244 X Sp. G

Sp. Gr.	2.67
K	126.14
	0.6250
h_s =	.3196
v_o	.3054
e_o	.9556
	1.1424

Time	500 lbs/□' Dial	%	1000 lbs/□' Dial	%	2000 lbs/□' Dial	%	4000 lbs/□' Dial	%	8000 lbs/□' Dial	%
0										
¼	35		26	87	18 49 13	69	27 56 71	82	22 69	73
½	34		30	100	19 54 73	73	28 60 75	85	23 74	77
1	34		30		20 58 77	77	29 68 78	88	25 77	83
2¼	34		30		21 64 81	81	29 73 83	88	25 81	83
4	34		30		22 71 84	85	29 78 86	88	25 83	83
9	37		30		24 80 89	92	29 83 90	88	25 87	83
16	40		30		25 86 92	96	30 86 92	91	25 88	83
25	41		30		25 86 92	96	31 89 94	94	25 90	83
36	41		30		25 89 94	96	32 94 95	97	25 91	83
49	42		30		25 89 94	96	32 93 95	97	25 91	83
64	44		30		25	96	32	97	26	87
2 hrs	44		30		25	96	32	97	26	87
3	44		36		25	96	33	100	27	90
4										
5			30		25	96	33	100	27	90
6			30		25	96	33	100	28	93
24			30		26		33		30	
		.0244		.0058		.0078		.0193		.0094
72	43	.0129		.0040		.0056		.0062		.0072
hv_o =										
hv_f =	.3011		.2981		.2955		.2922		.2892	
e_f =	.9421		.9327		.9246		.9143		.9049	
	1.0370		1.0168		.9894		.9534		.9189	

Total Defl. .0162

OPERATORS

COMMENTS

CONSOL SAMPLE

T......4

te 3-6-70- 3-13-70 mple No. C-6767-3

No. L-3598 Section RAMP "A" O-XING

n 8'8" 9'0" Station A 21+58 3'Rt Hole No. H-2

terial GREY SiLt SOME v/f SAND

Gross Weight	Tare	Net Weight	Can Number	Wet Weight	Dry Weight	Weight H_2O	% H_2O	Wet Density	Dry Density
464	149	315	57	60.70	41.31	19.39	46.9	104.1	70.9
		Consol	51	46.81	34.13	12.68	37.2		

mple Height = 4.0

= Wt. Dry Soil / K

= hv_o / h_s

= 47.244 X Sp. G

Time	500 lbs/□' Dial	%	1000 lbs/□' Dial	%	2000 lbs/□' Dial	%	4000 lbs/□' Dial	%	8000 lbs/□' Dial	%
0										
¼	54		15	14	16	12	33	19	36	22
½	66		17	18	19	14	38	22	51	31
1	83		20	21	27	20	53	30	61	37
2¼	125		21	22	41	30	80	46	86	52
4	173		34	35	55	41	101	56	101	61
9	251		48	50	75	56	124	71	122	73
16	316		58	60	90	67	132	76	129	77
25	342		63	66	96	71	137	79	134	80
36	357		66	69	105	78	143	82	136	81
49	368		71	74	107	79	145	83	140	84
64	373		75	78	108	80	148	85	141	84
2 hrs	379		82	85	114	84	156	90	147	86
3	382		85	89	117	87	159	91	153	92
4										
5			86	90	120	89	162	93	158	95
6			88	92	125	93	167	96	160	96
24			96		135		174		167	
72	406									
hv_o =										
hv_f =	.3097		.3001		.2866		.2692		.2525	
ef =	1.1274		1.0925		1.0433		.9800		.9192	

Sp. Gr.	2.63
K	124.25
	0.6250
h_s =	.2747
hv_o	.3503
e_o	1.2752

Total Defl. .0978

OPERATORS
COMMENTS
CONSOL
SAMPLE

T Grant

te 3-6-70 3-13-70 ple No. C-6757-3

No. L-3598 Section RAMP "A" O'XING

p 16'8" 17'0" Station A2d+31 5'Lt Hole No. H-1

terial GREY SILTY SAND SOME ORG. MAT

Gross Weight	Tare	Net Weight	Can Number	Wet Weight	Dry Weight	Weight H_2O	% H_2O	Wet Density	Dry Density
479	146	333	55	64.30	45.49	18.81	41.3	110.0	77.8
		Consol	49	50.70	37.43	13.27	35.5		

ple
ight = .410

= Wt. Dry Soil / K

= hv_o / h_s

= 47.244 X Sp. G

Time	500 lbs/□' Dial	%	1000 lbs/□' Dial	%	2000 lbs/□' Dial	%	4000 lbs/□' Dial	%	8000 lbs/□' Dial	%
0										
¼	120		25	52	49	66	69	68	68	69
½	134		30	63	51	69	73	72	71	72
1	154		32	67	56	74	79	77	74	75
2¼	167		33	69	60	81	87	85	78	79
4	173		35	73	64	87	88	86	81	82
9	180		39	81	67	91	92	90	86	87
16	182		39	81	67	91	92	90	87	88
25	185		39	81	68	92	95	93	89	90
36	188		40	83	69	93	97	95	91	92
49	189		40	83	69	93	97	95	92	93
64	190		41	85	70	95	98	96	93	94
2 hrs	192		42	87	71	96	99	97	94	95
3	192		42	87	71	96	100	98	96	97
4										
5			43	90	72	97	101	99	97	98
6			45	94	72	97	101	99	99	100
24			48		74		102		99	
72	193									
hv_o =										
hv_f =	.3090		.3042		.2968		.2866		.2767	
e_f =	1.0415		1.0253		1.0003		.9660		.9326	

Sp. Gr.	2.67
K	126.14
	0.6250
h_s =	.2967
hv_o	.3283
e_o	1.1065

Total Defl. .0516

OPERATORS
COMMENTS
CONSOL
SAMPLE

T...t

3598.

TIME - SETTLEMENT COMPUTATIONS

SH# ______ SECTION Ramp "A" O-XING A/E ______ STA ______ HOLE# ______

t	$\sqrt{t}$	500	1000	2000	4000	8000
1/4	0.5		76		85	
1/2	0.707		87		90	
1	1		89		91	
2-1/4	1.5		89		93	
4	2		89		94	
9	3		94		97	
16	4		96		97	
25	5		96		98	
36	6		96		98	
49	7		96		98	

%S	S	N_1	$t=N\left(\frac{.405d^2}{c}\right)$	N	$t=N\left(\frac{.405d^2}{c}\right)$
10		0.02	yrs		
20		0.08			
30		0.17			
40		0.31			
50		0.49			
60		0.71			
70		1.00	.0496	0.009	
80		1.40	.0695	0.013	
90		2.09	.1040	0.019	
100		∞	∞	∞	∞

SAMPLE NO. C-6759-2, C-6771-2, C-6772-2 LAYER 2

$$\frac{0.405\,d^2}{c} = \frac{0.405\left(\frac{14}{2}\right)^2}{400} = 0.0496$$

$$= \frac{.405\left(\frac{6}{2}\right)^2}{400} = 0.009$$

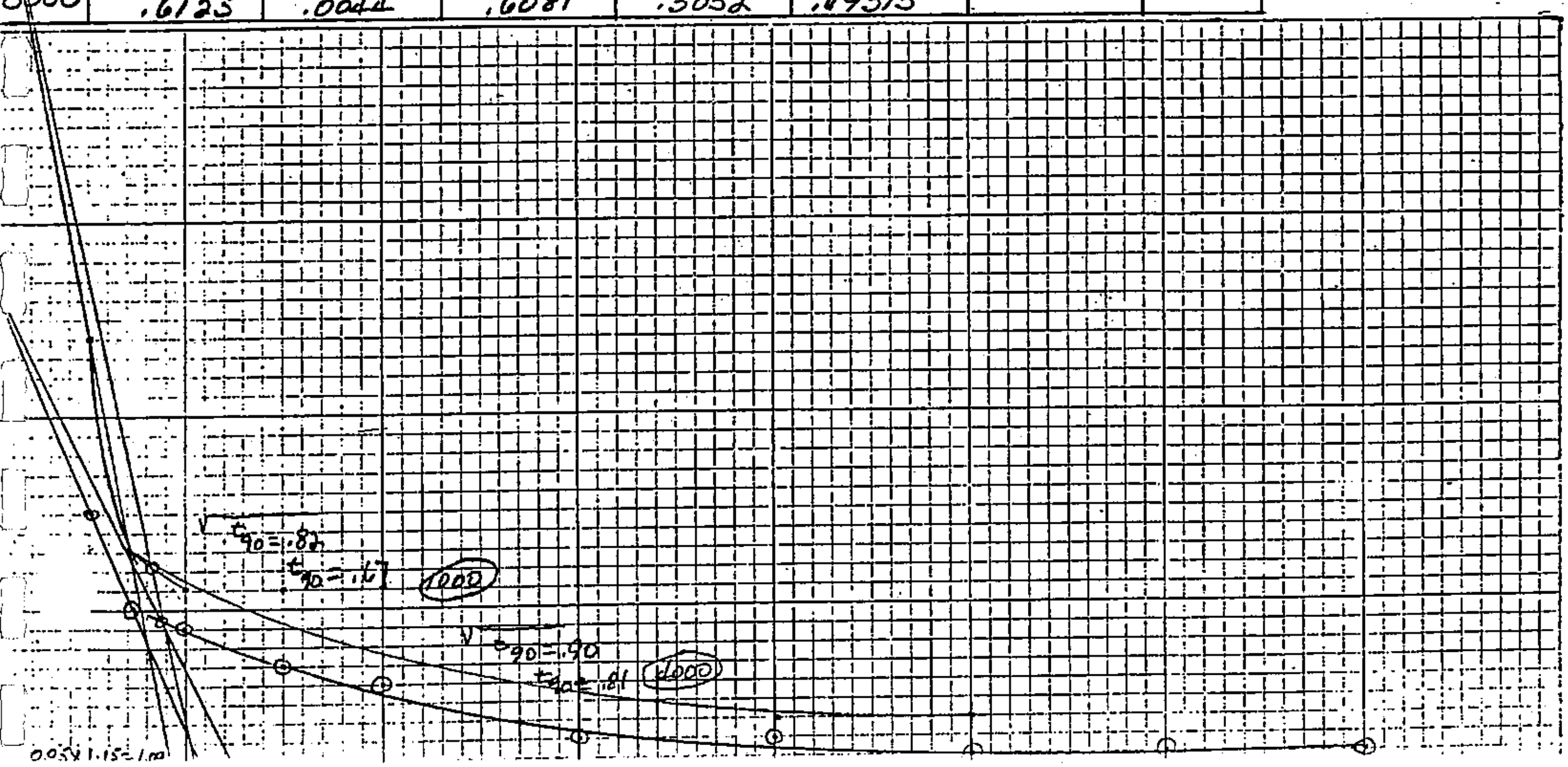
LOAD	h_i (in)	Δh	h_f	$\frac{h_i+h_f}{4}$	$\left(\frac{h_i+h_f}{4}\right)^2$	t_{90} (min)	$c = \frac{3095}{t_{90}}\left(\frac{h_i+h_f}{4}\right)^2$ (ft²/yr)
500	0.6250	.0043	.6207	.3114	.09697	-	
1000	.6207	.0022	.6185	.3098	.09598	0.67	[illegible]
2000	.6185	.0034	.6151	.3084	.09511		—
4000	.6151	.0026	.6125	.3069	.09419	0.81	361[illegible]
8000	.6125	.0044	.6081	.3052	.09315		

c= @

c= @

c= @

c= @



te 3-6-70. 3-13-70 ple No. C-6771-2

No. L-3598 Section RAMP "A" O'-XING

o. 18'8" 19'0" Station A 21+58 3' RT Hole No. H-2

terial GREY fairly clean SAND

Gross ight	Tare	Net Weight	Can Number	Wet Weight	Dry Weight	Weight H_2O	% H_2O	Wet Density	Dry Density
484	145	~~339~~ 357	58	79.72	61.94	17.78	28.7	118.0	91.7
		Consol	52	56.43	44.57	11.86	26.6		

ple ight = 3,8

= Wt. Dry Soil / K

= hv_o / h_s

= 47.244 X Sp. G

Time	500 lbs/□' Dial	%	1000 lbs/□' Dial	%	2000 lbs/□' Dial	%	4000 lbs/□' Dial	%	8000 lbs/□' Dial	%
0										
¼	45		16 76	80	29	100	22 85	88	35	88
½	45		17 87	85	29		23 90	92	35	88
1	45		17 89	85	29		24 91	96	35	88
2¼	46		17 89	85	29		24 93	96	35	88
4	46		17 89	85	29		24 94	96	35	88
9	46		20 94	100	29		25 97	100	36	90
16	46		20 96		29		25 97		38	95
25	46		20 96		29		25 98		38	95
36	46		20 96		29		25 98		39	98
49	46		20 96		29		25 98		39	98
64	46		20		29		25		39	98
2 hrs	46		20		29		25		39	98
3	46		20		29		25		39	98
4										
5			20		29		25		39	98
6			20		29		25		40	100
24			20		29		25		40	
72 -	46	0013		0022		0031		0026		0041
hv_o =										
hv_f =	.2671		.2651		.2622		.2597		.2557	
ef =	.7560		.7502		.7421		.7351		.7237	
	.7212		.7152		.7058		.6988		.6866	

. Gr.	2.67
K	126.12
	0.6250
s =	.3533
o	.2717
o	.7690
	.7332

Total Defl. .0160

OPERATORS
COMMENTS
CONSOL
SAMPLE

T...t

ate 3-6-70 3-13-70 ample No. C-6759-2

No. L-3598 Section RAMP "A" O-XING

61'4" 61'8" Station A24+31 5' LT Hole No. H-1

aterial GREY fairly clean SAND

Gross Weight	Tare	Net Weight	Can Number	Wet Weight	Dry Weight	Weight H_2O	% H_2O	Wet Density	Dry Density
496	146	~~350~~ 369	56	82.28	70.23	17.05	24.3	121.9	98.1
		Consol	50	56.54	44.00	12.54	28.5		

Sample Weight = 3.8

W_s = Wt. Dry Soil / K

= hv_o / h_s

K = 47.244 X Sp. G

Sp. Gr.	2.64
K	124.72
	0.6250
h_s =	.3528
hv_o	.2722
e_o	.7715

Time	500 lbs/□' Dial	%	1000 lbs/□' Dial	%	2000 lbs/□' Dial	%	4000 lbs/□' Dial	%	8000 lbs/□' Dial	%
0										
¼	39		11	61	30	100	13	87	28	82
½	39		15	83	30		14	93	30	88
1	39		16	89	30		14	93	31	91
2¼	39		16	89	30		14	93	31	91
4	40		16	89	30		14	93	31	91
9	41		16	89	30		15	100	32	94
16	41		17	94	30		15		32	94
25	41		17	94	30		15		32	94
36	41		17	94	30		15		32	94
49	42		17	94	30		15		32	94
64	42		17	94	30		15		32	94
2 hrs	42		18	100	30		15		33	97
3	42		18	100	30		15		33	97
4										
5			18	100	30		15		33	97
6			18	100	30		15		33	97
24			18		30		15		34	
72	42									
hv_o =										
hv_f =	.2680		.2662		.2632		.2617		.2583	
ef =	.7596		.7515		.7460		.7418		.7321	

Total Defl. .0139

OPERATORS
COMMENTS
CONSOL
SAMPLE

T [illegible]

ate 3-6-70- 3-13-70 Sample No. C-6772-2

No. L-3598 Section RAMP "A" O-XING

21'6" 21'10" Station A21+58 3'RT Hole No. H-2

aterial Grey Silty Sand

Gross Weight	Tare	Net Weight	Can Number	Wet Weight	Dry Weight	Weight H_2O	% H_2O	Wet Density	Dry Density
508	145	363	59	115.32	91.70	23.62	25.8	119.9	95.3
		Consol	53	59.11	47.34	11.77	24.9		

mple
eight = 4.0

$= \frac{\text{Wt. Dry Soil}}{K}$

$= hv_o \ / \ h_s$

= 47.244 X Sp. G

Sp. Gr.	2.66
K	125.67
	0.6250
hs =	.3767
hv_o	.2483
e_o	.6591

Time	500 lbs/□'		1000 lbs/□'		2000 lbs/□'		4000 lbs/□'		8000 lbs/□'	
	Dial	%	Dial	%	Dial	%	Dial	%	Dial	%
0										
¼	37		24	84	38	91	30	81	48	81
½	37		26	93	38	91	31	84	50	85
1	37		26	93	38	91	31	84	51	86
2¼	37		26	93	38	91	33	89	54	91
4	37		26	93	38	91	34	92	56	93
9	39		26	93	39	93	34	92	55	93
16	41		26	93	40	95	34	92	55	93
25	41		26	93	40	95	35	95	56	95
36	41		26	93	40	95	35	95	56	95
49	41		26	93	40	95	35	95	56	95
64	41		26	93	40	95	35	95	56	95
2 hrs	41		26	93	40	95	36	97	57	97
3	41		27	97	40	95	36	97	58	98
4										
5			27	97	40	95	36	97	58	98
6			27	97	40	95	37	97	58	98
24			28		42		37		59	
72	42									
hv_o =										
hv_f =	.2441		.2413		.2371		.2332		.2275	
ef =	.6480		.6406		.6292		.6196		.6039	

Total Defl. .0208

OPERATORS
COMMENTS
CONSOL
SAMPLE

3598

TIME - SETTLEMENT COMPUTATIONS

SH# ______ SECTION Ramp "A" O-xing ______ STA ______ HOLE# ______

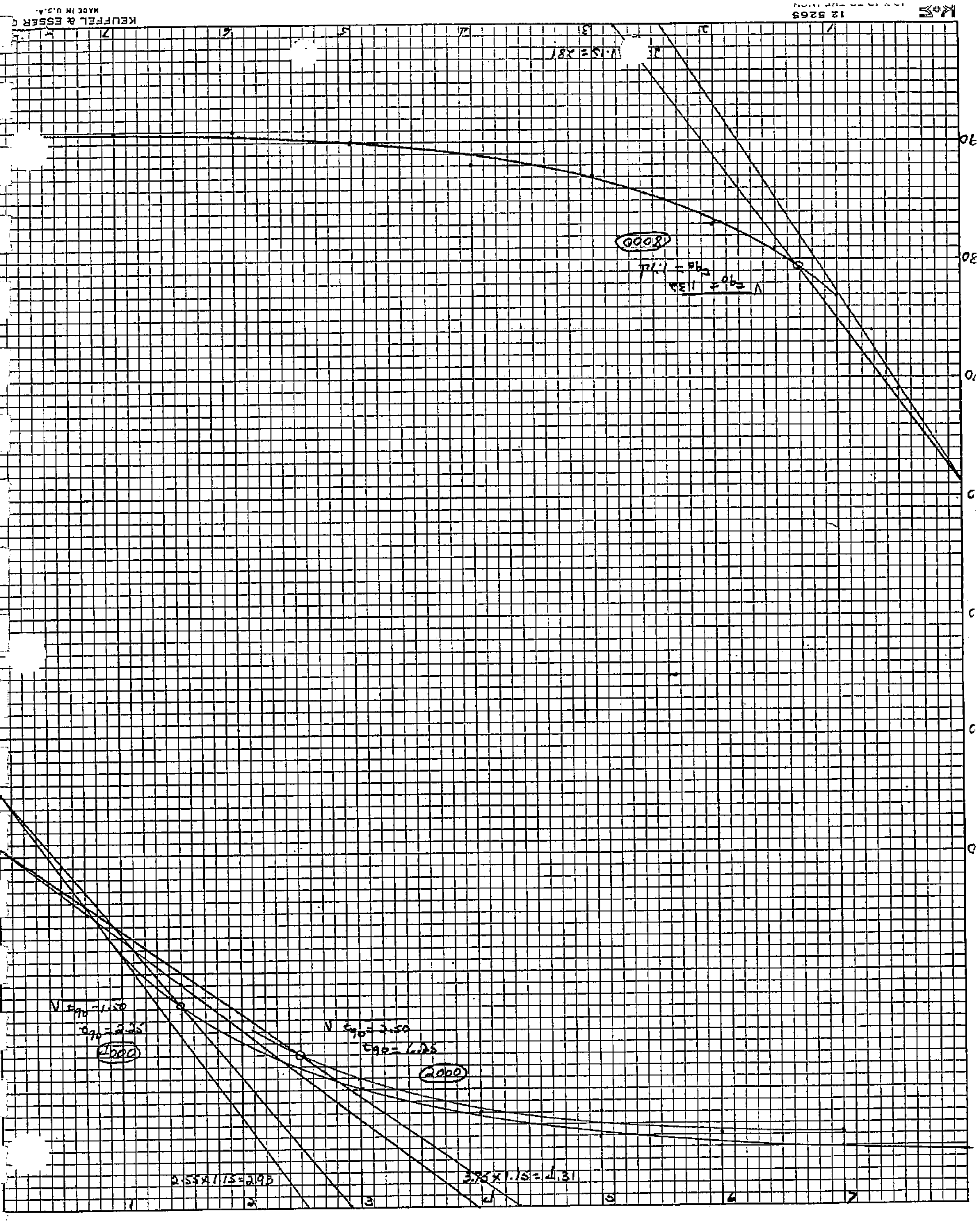
t	$\sqrt{t}$	500	1000	2000	4000	8000
/4	0.5			73	71	69
/2	0.707			75	75	74
1	1			77	78	77
1/4	1.5			81	83	81
4	2			84	86	83
9	3			89	90	87
16	4			92	92	88
25	5			92	94	90
36	6			94	95	91
9	7			94	95	91

%S	S	N_1	$t = N_1\left(\frac{.405d^2}{c}\right)$	N	$t = N\left(\frac{.405d^2}{c}\right)$
10		0.02	yrs		
20		0.08			
30		0.17			
40		0.31			
50		0.49			
60		0.71			
70		1.00			
80		1.40			
90		2.09			
100		∞	∞	∞	∞

SAMPLE NO. Ave All Samples LAYER ______ $\frac{0.405\,d^2}{c} =$

LOAD	h_i (in)	Δh	h_f	$\frac{h_i+h_f}{4}$	$\left(\frac{h_i+h_f}{4}\right)^2$	t_{90} (min)	$c = \frac{3095}{t_{90}}\left(\frac{h_i+h_f}{4}\right)^2$ (ft²/yr)
	0.6250	.0129	.6121	.3093	.0957		
00	.6121	.0040	.6081	.3051	.0931	-	
00	.6081	.0056	.6025	.3027	.0916	6.25	45.4
000	.6025	.0064	.5961	.2997	.0898	2.25	123.5
00	.5961	.0072	.5889	.2963	.0878	1.74	156.2

c=	@
c=	@
c=	@
c=	@



√t90 = 1.50
t90 = 2.25
1000
√t90 = 2.50
t90 = 6.25
2000
2.55 × 1.15 = 2.93
3.75 × 1.15 = 4.31
1
2
3
4
5
6
7

26.66 (Rev. 5-67)

NE Ramp

WASHINGTON
STATE HIGHWAY COMMISSION
DEPARTMENT OF HIGHWAYS

Original to Materials Engineer
Copy to Bridge Engineer
Copy to District Engineer
Copy to

LOG OF TEST BORING

S.H. ______ S.R. 167 Section West Auburn Interchange, Pier #1 Job No. L-3598
Hole No. H-2 Sub Section Ramp A Overcrossing (A/E) Cont. Sec. 176501
Station A 21+58 Offset 3' Rt. of ℄ Contour Ground El. 63'0"
Type of Boring Chop and Drive Casing 101' x 3" W.T. El. 55'0"
Inspector LeRoy R. Sampson Date February 3, 1970 Sheet 1 of 5

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
			B C	1'6" of swamp water over ground surface.
			D U-1	PEAT - Very soft, saturated, brown.
				Logs or wood fragments possible through
	1/18"		1/18" Std. Pen. 2	the depth of this boring.
				SILT WITH SAND LENSES - Very soft, gray
				silt with pockets of moist, brown, organic
				silt and 6" ± lenses of fine, wet, dark
			A B C U-3	gray sand.
			D E	SILT - Soft, gray, moist.
			2 4 Std. Pen.	
	8		4 6 4	FINE SAND - Loose to slightly compact, wet,
			A U-5	dark gray
			2 9 Std. Pen.	
	20		11 11 6	
			10 9 Std. Pen.	SILT - Slightly compact (possibly soft), brownish
	17		8 8 7	gray, moist, small pieces of peat scattered
			B C U-8	through
				SILTY, VERY FINE SAND - Slightly compact, wet, gray

Hole No. H-2 Sub Section Ramp A Overcrossing (A/E) Sheet 2 of 5

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
			A B C U-9	
	14		4 5 9 18 Std. Pen. 10	SANDY GRAVEL - Dense, gray, wet, fine to coarse sand and fine to coarse gravel, a trace of silt.
	39		25 20 19 14 Std. Pen. 11	
	45		17 20 25 28 Std. Pen. 12	
	40 17		19 21 9 8 Std. Pen. 13	SANDY GRAVEL - Slightly compact, gray, fine to coarse sand and fine to coarse gravel.
	48		30 22 26 35 Std. Pen. 14	SANDY GRAVEL - Compact to very dense, gray, wet, fine to coarse sand and fine to coarse gravel and scattered cobbles, a trace of silt.

-le No. H-2 Sub Section Ramp A Overcrossing (A/E) Sheet 3 of 5

BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
29		23, 17, 12, 11 — Std. Pen. 15	
71/9"		25, 41, 30/3" — Std. Pen. 16	
			Cobbles probably not present beyond 54'.
38		14, 17, 21, 26 — Std. Pen. 17	
69		34, 30, 39 — Std. Pen. 18	FINE AND MEDIUM SAND - Dense, dark gray, a trace of silt. (This stratum heaves.)
54		14, 28, 26, 21 — Std. Pen. 19	SILTY, SANDY GRAVEL - Loose, gray, moist, fine to coarse sand and gravel.
			LENSES OF GRAY CLAY 68' to 70'.

Hole No. H-2 Sub Section Ramp A Overcrossing (A/E) Sheet 4 of 5

(DEP)TH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
			A D H I U-20	
5	12		5 6 6 6 Std. Pen. 21	
)				
	8		9 4 4 10 Std. Pen. 22	
5				
	11		7 5 6 6 Std. Pen. 23	
0				
	17		25 9 8 8 Std. Pen. 24	
5				

Hole No. H-2 Sub Section Ramp A Overcrossing (A/E) Sheet 5 of 5

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	34		22 20 14 13 Std. Pen. 25	
100				
	25		16 10 15 9 Std. Pen. 26	
	18		10 9 9 Std. Pen. 27	
105				TEST BORING STOPPED AT 104'6"

1
2
3

H-1

TRIAXIAL TEST DATA

Date 2-18-70 Sample No. C-67513

Job No. L-3598 Section Ramp "A" O'-xing

Depth 0'0" 1'-8" Station A24+31 Offset 5' LT

Material Brown Peat - Some Silt

Gross Wt.	Tare	Net Wt.	Can No.	Wet Wt.	Dry Wt.	Wt. H_2O	% H_2O	Wet Density	Dry Density
352	144	208	13	68.10	17.09	51.01	29.8	68.7	52.9

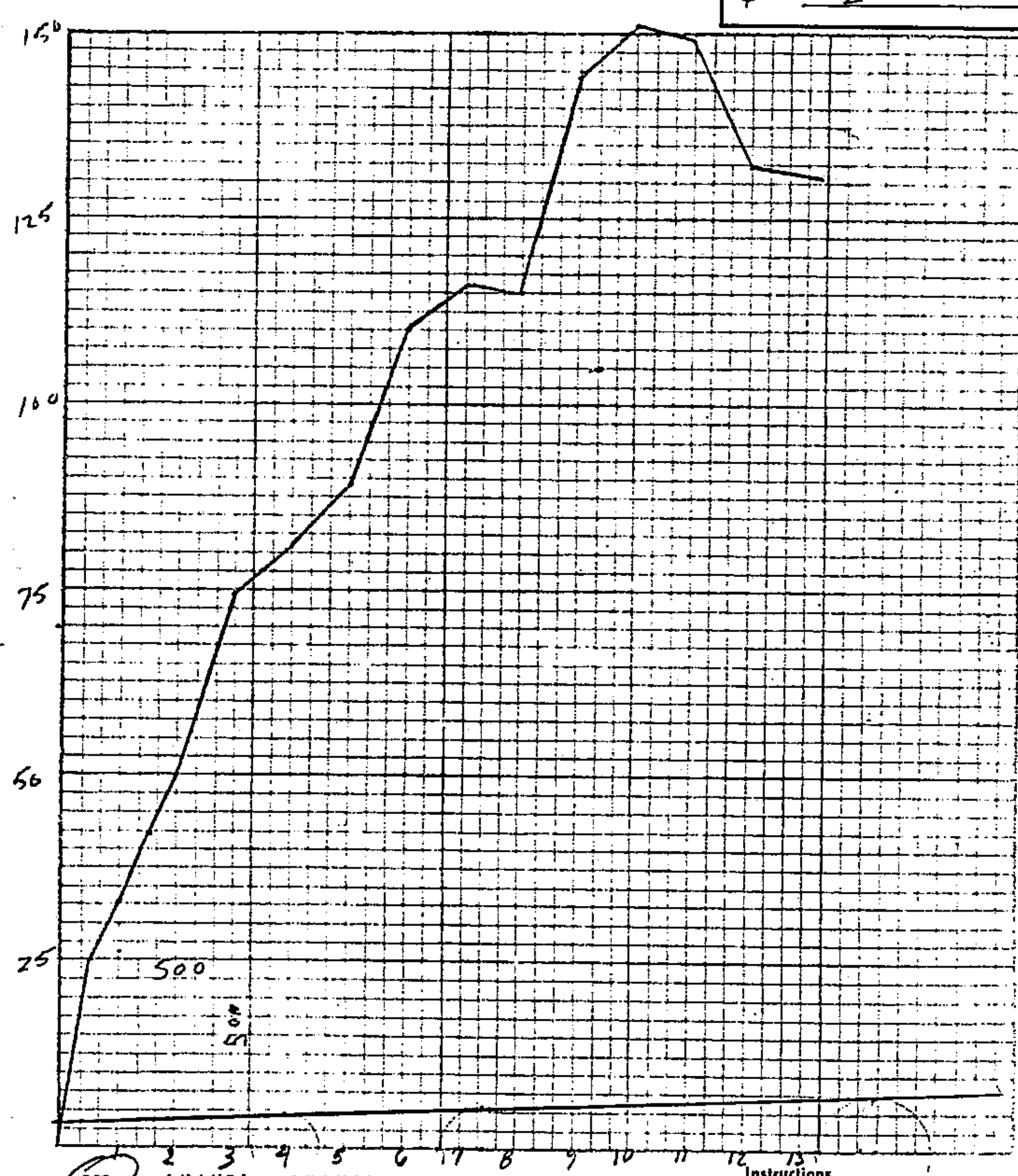
σ_3	$\frac{1}{2}(\sigma_1-\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)-u$
500	89	589	
1000	115	1115	
2000	149	2149	

Sample Height = 4.0

C = 75 #/Sq. Ft.

ϕ = 2° Deg.

Strain %	Dial Reading in inch × 10⁻⁴	$\frac{1}{2}(\sigma_1-\sigma_3)$ #/sq. ft.	Strain inch × 10⁻³	Press. Gauge #/sq. in.
0			Initial	
0.5	3	25		
1.0	4	33		
1.5	5	42		
2.0	6	50		
3.0	9	74		
4.0	10	81		
[illegible]	11	89	500	
[illegible]	14	110		
7.0	15	116		
8.0	15	115	1000	
9.0	19	144		
10.0	20	151		
11.0	20	149	2000	
12.0	18	132	500	
13.0	18	131		
14.0				
15.0				
16.0				
17.0				
18.0				
19.0				
20.0				
21.0				
22.0				
23.0				
24.0				
25.0				



Stress #/ sq. ft.

Type Machine ✓ KW WS ST UW(1) UW(2)

Type Test	Stage ✓	Uncon.	Con. Drained	Con. Undrained	Consol	

Instructions
Show Stress-Strain in Red.
Show Mohr's diagram in Black.

Operator TERRY

1
2
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4

H1

TRIAXIAL TEST DATA

2-18-70 Sample No. C-67534

Job No. L-3598 Section RAMP "A" O-XING

Depth 6'0" 7'8" Station A24 +31 Offset 5' LT

Material Grey Fairly Clean Fine Sand

Gross Wt.	Tare	Net Wt.	Can No.	Wet Wt.	Dry Wt.	Wt. H_2O	% H_2O	Wet Density	Dry Density
499	145	~~354~~ 363	14	110.04	85.68	24.36	28.4	119.9	93.4

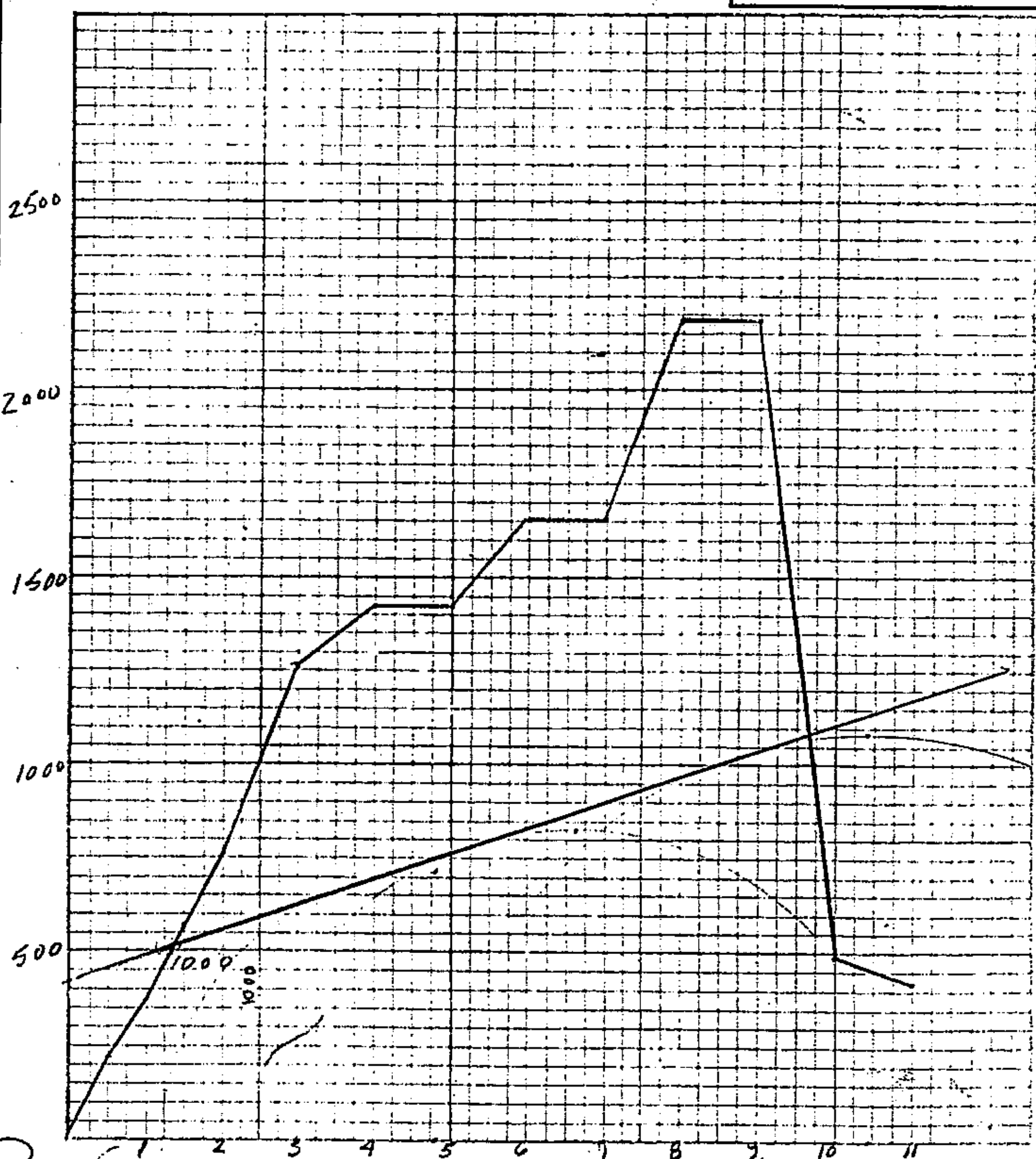
σ_3	$\frac{1}{2}(\sigma_1-\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)-u$
500	1420	1920	
1000	1650	2650	
2000	2180	4180	

Sample Height = 3.9

C = 850 #/Sq. Ft.

ϕ = 19° Deg.

Strain %	Dial Reading in inch × 10	$\frac{1}{2}(\sigma_1-\sigma_3)$ #/sq. ft.	Strain inch × 10	Press. Gauge #/sq. in.
0			Initial	
0.5	9	224		
1.0	15	370		
1.5	23	565		
2.0	31	760		
3.0	52	1260		
4.0	59	1420		
[illegible]	60	1420	500	
	70	1650		
7.0	71	1650	1000	
8.0	95	2180		
9.0	96	2180	2000	
10.0	22	490	500	
11.0	19	420		
12.0				
13.0				
14.0				
15.0				
16.0				
17.0				
18.0				
19.0				
20.0				
21.0				
22.0				
23.0				
24.0				
25.0				



Instructions
Show Stress-Strain in Red.
Show Mohr's diagram in Black.

Type Machine ~~KW~~ WS ST UW(1) UW(2)

Type Test					
~~Stage~~	Uncon.	Con. Drained	Con. Undrained	Consol	

Operator TERRY

H-1

TRIAXIAL TEST DATA

2-18-70 Sample No. C-6755-1

Job No. L-3598 Section RAMP "A" O-XING

Depth 11'2" 11'10" Station A24 + 31 Offset 5' LT

Material Grey Fairly Clean Sand

Gross Wt.	Tare	Net Wt.	Can No.	Wet Wt.	Dry Wt.	Wt. H_2O	% H_2O	Wet Density	Dry Density
396	144	~~252~~ 265	15	122.60	97.20	254	26.1	87.6	69.5

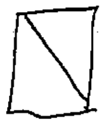
σ_3	$\frac{1}{2}(\sigma_1-\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)-u$
500	1140	1640	
1000	1340	2340	
2000	1530	3530	

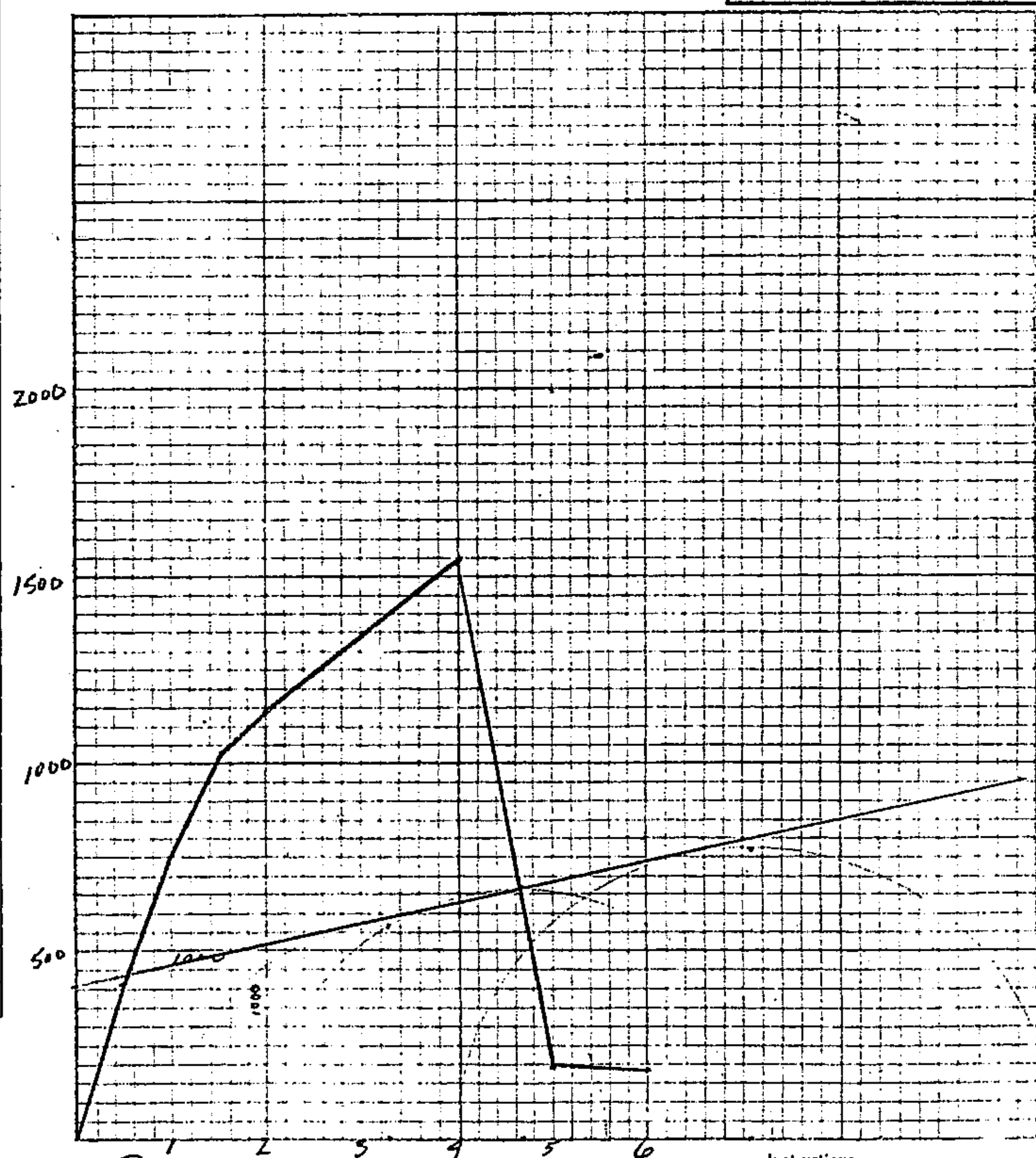
Strain %	Dial Reading in inch × 10	$\frac{1}{2}(\sigma_1-\sigma_3)$ #/sq. ft.	Strain inch × 10	Press. Gauge #/sq. in.
0			Initial	
0.5	50	420		
1.0	90	750		
1.5	122	1020		
2.0	140	1140	500	
3.0	165	1340	1000	
4.0	190	1530	2000	
[illegible]0	25	199	500	
[illegible]	25	196		
7.0				
8.0				
9.0				
10.0				
11.0				
12.0				
13.0				
14.0				
15.0				
16.0				
17.0				
18.0				
19.0				
20.0				
21.0				
22.0				
23.0				
24.0				
25.0				

Sample Height = 3.8

C = 810 #/Sq. Ft.

ϕ = 13° Deg.





Type Machine ✓ KW WS (ST) UW(1) UW(2)

Type Test: Stage ✓ | Uncon. | Con. Drained | Con. Undrained | Consol

Instructions
Show Stress-Strain in Red.
Show Mohr's diagram in Black.

Operator TERRY

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11-1

TRIAXIAL TEST DATA

Date 2-18-70 — Sample No. C-6757-1

Job No. L-3598 — Section RAMP "A" O-XING

Depth 16'0" 17'4" — Station A 24 + 31 — Offset 5' LT.

Material Grey Sandy Fine Silt - Some Org Mat

Gross Wt.	Tare	Net Wt.	Con No.	Wet Wt.	Dry Wt.	Wt. H_2O	% H_2O	Wet Density	Dry Density
490	149	341	16	108.26	74.72	33.54	44.9	112.7	77.8

σ_3	$\frac{1}{2}(\sigma_1-\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)-u$
500	214	714	
1000	293	1293	
2000	350	2350	

Sample Height = 4.0

C = 200 #/Sq. Ft.

ϕ = 4° Deg.

Strain %	Dial Reading in inch × 10⁻⁴	$\frac{1}{2}(\sigma_1-\sigma_3)$ #/sq. ft.	Strain inch × 10⁻³	Press. Gauge #/sq. in.
0			Initial	
0.5	9	75		
1.0	11	92		
1.5	12	100		
2.0	14	114		
3.0	20	162		
4.0	25	200		
	27	214	500	
	31	244		
7.0	37	289		
8.0	38	293	1000	
9.0	42	320		
10.0	46	350	2000	
11.0	40	300	500	
12.0	41	305		
13.0				
14.0				
15.0				
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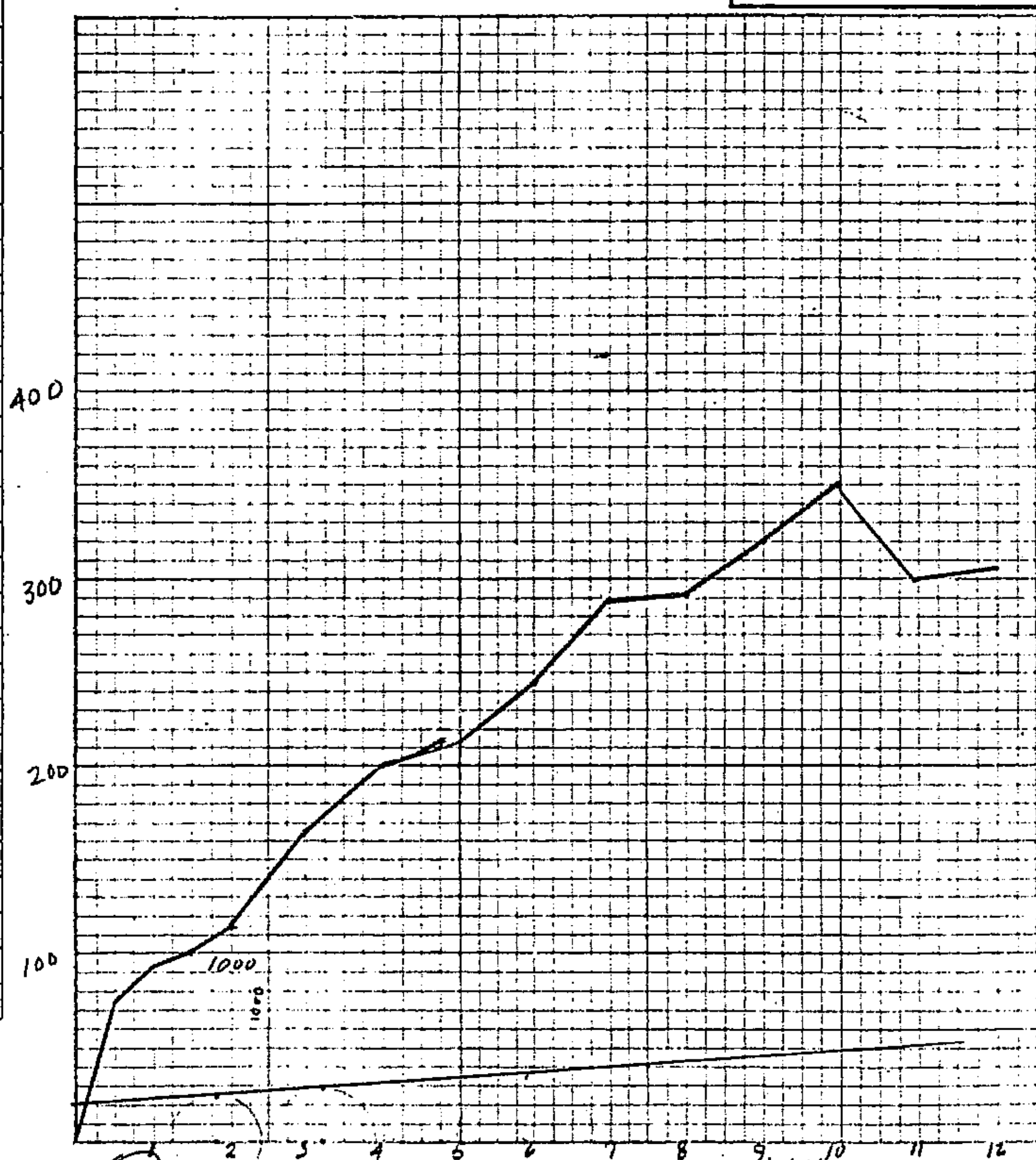
24.0				
25.0				



Instructions
Show Stress-Strain in Red.
Show Mohr's diagram in Black.

Type Machine ✓ KW WS ST UW(1) UW(2)

Type Test	Stage	Uncon.	Con. Drained	Con. Undrained	Consol	

Operator TERRY

1
2

H-1

TRIAXIAL TEST DATA

2-18-70 | Sample No. C-6759-1

Job No. L-3598 | Section RAMP "A" O-XING

Depth 61'0" 61'8" | Station A24+31 | Offset 5' LT

Material Grey Fairly Clean Coarse Sand

Gross Wt.	Tare	Net Wt.	Can No.	Wet Wt.	Dry Wt.	Wt. H_2O	% H_2O	Wet Density	Dry Density
518	147	371	17	110.25	89.83	20.42	22.7	122.6	99.9

σ_3	$\frac{1}{2}(\sigma_1-\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)-u$
500	410	910	
1000	485	1485	
2000	725	2725	

Sample Height = 4.0

C = 250 #/Sq. Ft.

ϕ = 10° Deg.

Strain %	Dial Reading in Inch × 10⁻⁴	$\frac{1}{2}(\sigma_1-\sigma_3)$ #/sq. ft.	Strain Inch × 10⁻³	Press. Gauge #/sq. in.
0			Initial	
0.5	30	255		
1.0	40	335		
1.5	46	385		
2.0	48	390		
3.0	50	410	500	
4.0	60	485	1000	
5.0	90	725	2000	
6.0	23	182	500	
7.0	20	156		
8.0				
9.0				
10.0				
11.0				
12.0				
13.0				
14.0				
15.0				
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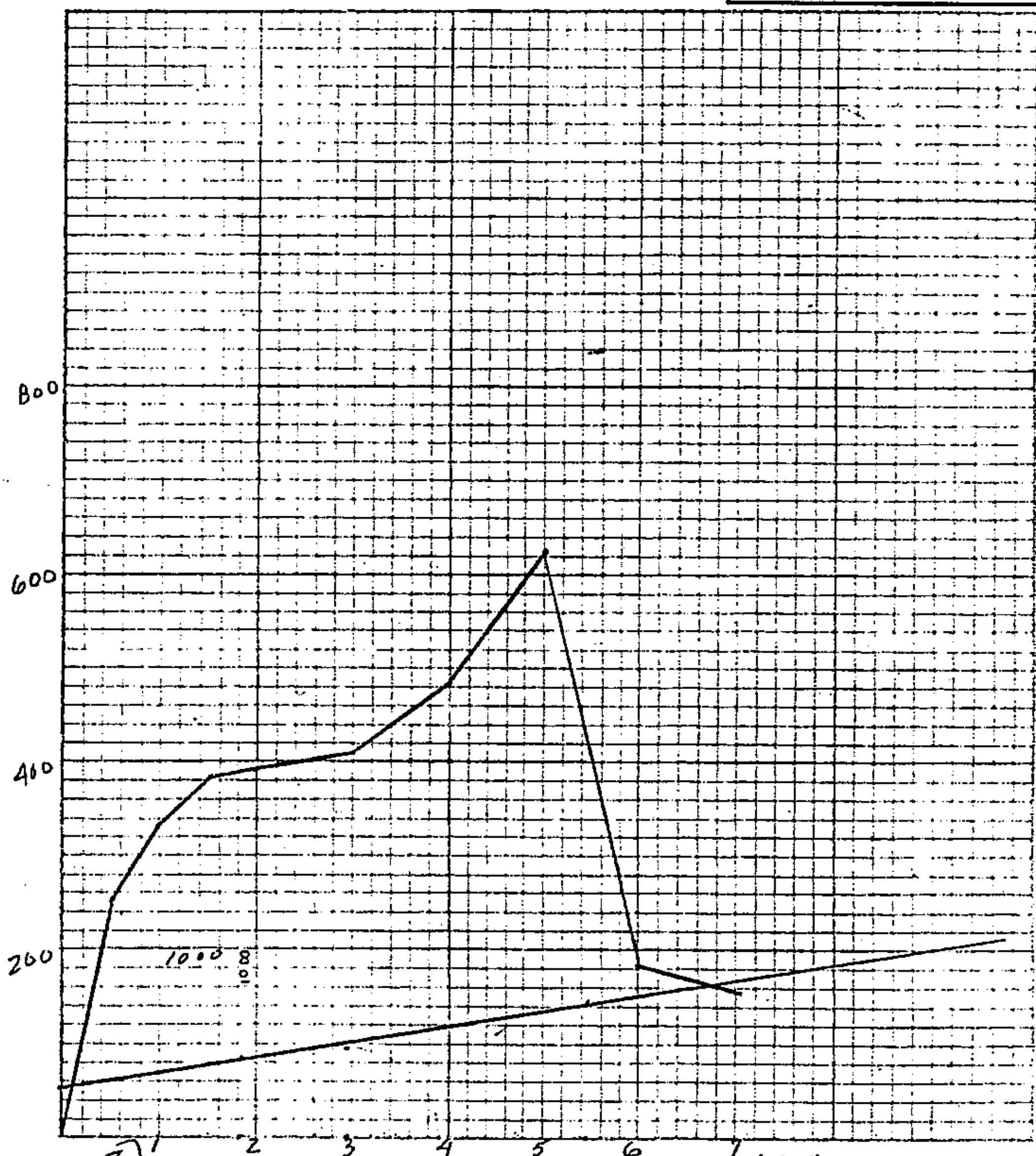
25.0				



4-11-72

Type Machine KW WS ST UW(1) UW(2)

Type Test	Stage	Uncon.	Con. Drained	Con. Undrained	Consol	

Instructions
Show Stress-Strain in Red.
Show Mohr's diagram in Black.

Operator TERRY

H-1

1
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TRIAXIAL TEST DATA

.. 2-18-70 — Sample No. C-6761-1

Job No. L-3598 — Section RAMP "A" O'XING

Depth 66'0" 67'8" — Station Add+31 — Offset 5' LT.

Material Grey Compact Silty Sandy Grav to ½" Sub Rd

Gross Wt.	Tare	Net Wt.	Can No.	Wet Wt.	Dry Wt.	Wt. H_2O	% H_2O	Wet Density	Dry Density
545	145	400	18	117.20	99.14	18.06	18.2	132.2	111.8

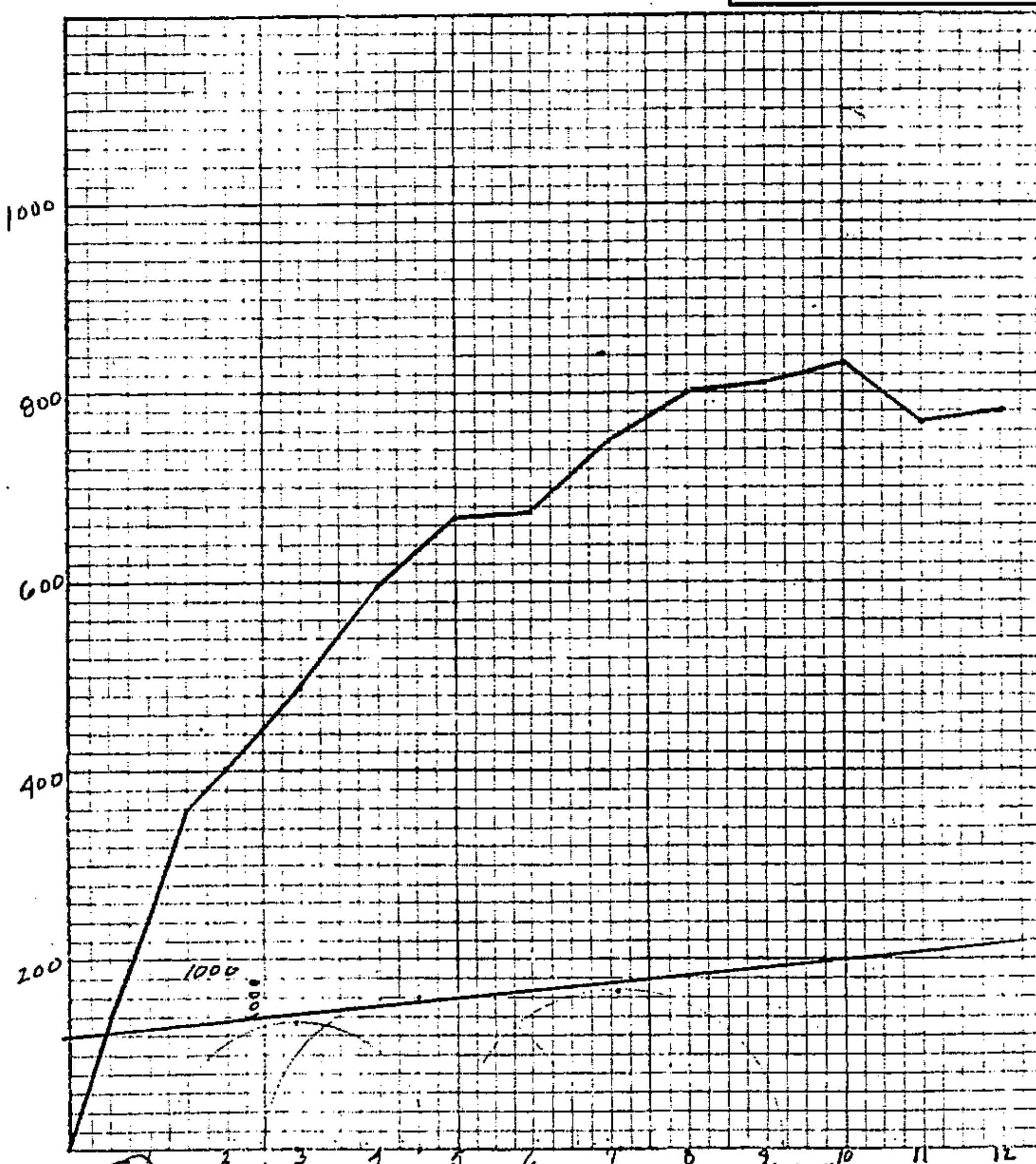
σ_3	$\frac{1}{2}(\sigma_1-\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)-u$
500	675	1175	
1000	800	1800	
2000	830	2830	

Strain %	Dial Reading in Inch × 10⁻⁴	$\frac{1}{2}(\sigma_1-\sigma_3)$ #/sq. ft.	Strain Inch × 10⁻³	Press. Gauge #/sq. in.
0			Initial	
0.5	17	140		
1.0	30	246		
1.5	43	360		
2.0	48	400		
3.0	60	490		
4.0	73	595		
[illegible]	83	670		
[illegible]	85	675	500	
7.0	95	750		
8.0	105	800	1000	
9.0	107	810		
10.0	110	830	2000	
11.0	103	770	500	
12.0	164	780		
13.0				
14.0				
15.0				
16.0				
17.0				
18.0				
19.0				
20.0				
21.0				
22.0				
23.0				
24.0				
25.0				

Sample Height = 4.0

C = 600 #/Sq. Ft.

ϕ = 6° Deg.



% Strain

Instructions
Show Stress-Strain in Red.
Show Mohr's diagram in Black.

Type Machine KW WS ST UW(1) UW(2)

Type Test	Stage	Uncon.	Con. Drained	Con. Undrained	Consol	

Operator TERRY

1
2
3

H-2

TRIAXIAL TEST DATA

2-18-70 Sample No. C-6765-3

Job No. L-3598 Section RAMP "A" OVERCROSSING

Depth 0'6" 1'6" Station A21 +58 Offset 3' RT ℄

Material Brown Peat

Gross Wt.	Tare	Net Wt.	Can No.	Wet Wt.	Dry Wt.	Wt. H_2O	% H_2O	Wet Density	Dry Density
330	143	187	19	61.27	60.08	119	198.1	61.8	20.7

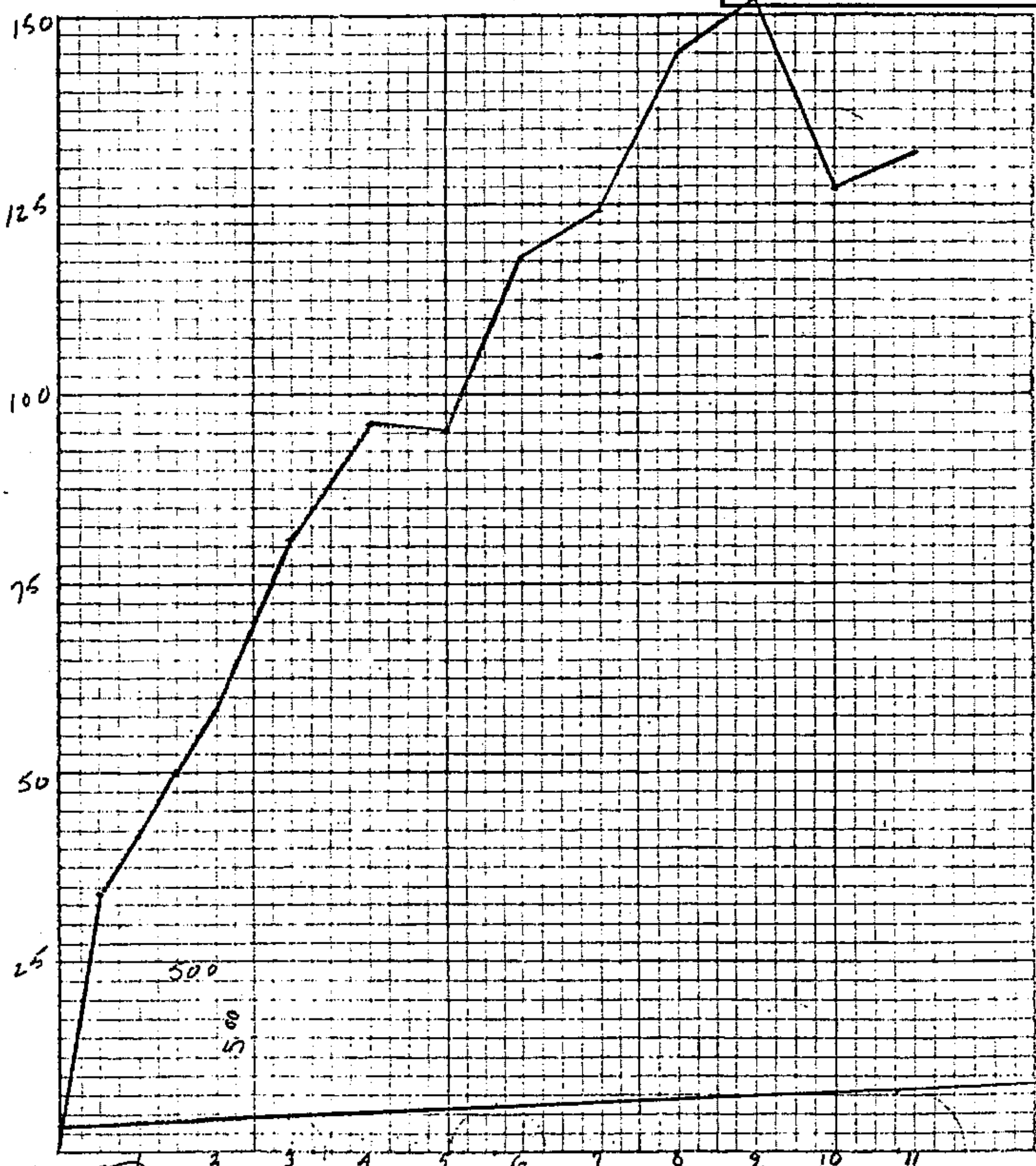
σ_3	$\frac{1}{2}(\sigma_1-\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)-u$
500	95	595	
1000	124	1124	
2000	153	2153	

Strain %	Dial Reading in Inch × 10	$\frac{1}{2}(\sigma_1-\sigma_3)$ #/sq. ft.	Strain Inch × 10	Press. Gauge #/sq. in.
0			Initial	
0.5	4	34		
1.0	5	42		
1.5	6	50		
2.0	7	58		
3.0	10	81		
4.0	12	96		
[illegible]	12	95	500	
	15	118		
7.0	16	124	1000	
8.0	19	145		
9.0	20	153	2000	
10.0	17	127	500	
11.0	18	132		
12.0				
13.0				
14.0				
15.0				
16.0				
17.0				
18.0				
19.0				
20.0				
21.0				
22.0				
23.0				
24.0				
25.0				

Sample Height = 4.0

C = 70 #/Sq. Ft.

ϕ = 3° Deg.



Instructions
Show Stress-Strain in Red.
Show Mohr's diagram in Black.

Type Machine KW WS ST UW(1) UW(2)

Type Test	Stage	Uncon.	Con. Drained	Con. Undrained	Consol	

Operator TERRY

H-2

TRIAXIAL TEST DATA

Date. 2-18-70 Sample No. C-6767-1

Job No. L-3598 Section RAMP "A" O'XING

Depth 8'0" 9'8" Station A21+58 Offset 3' RT ℄

Material Grey Fine Sandy Silt - Some Org Mat

Gross Wt.	Tare	Net Wt.	Can No.	Wet Wt.	Dry Wt.	Wt. H_2O	% H_2O	Wet Density	Dry Density
446	154	~~292~~ 300	20	109.38	66.67	42.71	64.1	99.1	60.2

σ_3	$\frac{1}{2}(\sigma_1-\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)-u$
500	104	604	
1000	178	1178	
2000	243	2243	

Sample Height = 3.9

C = 75 #/Sq. Ft.

ϕ = 5 Deg.

Strain %	Dial Reading in inch × 10	$\frac{1}{2}(\sigma_1-\sigma_3)$ #/sq. ft.	Strain inch × 10	Press. Gauge #/sq. in.
0			Initial	
0.5	5	42		
1.0	7	59		
1.5	9	75		
2.0	11	91		
3.0	12	97		
4.0	13	104	500	
	18	143		
6.0	22	173		
7.0	23	178	1000	
8.0	28	215		
9.0	32	243	2000	
10.0	21	158	500	
11.0	23	170		
12.0				
13.0				
14.0				
15.0				
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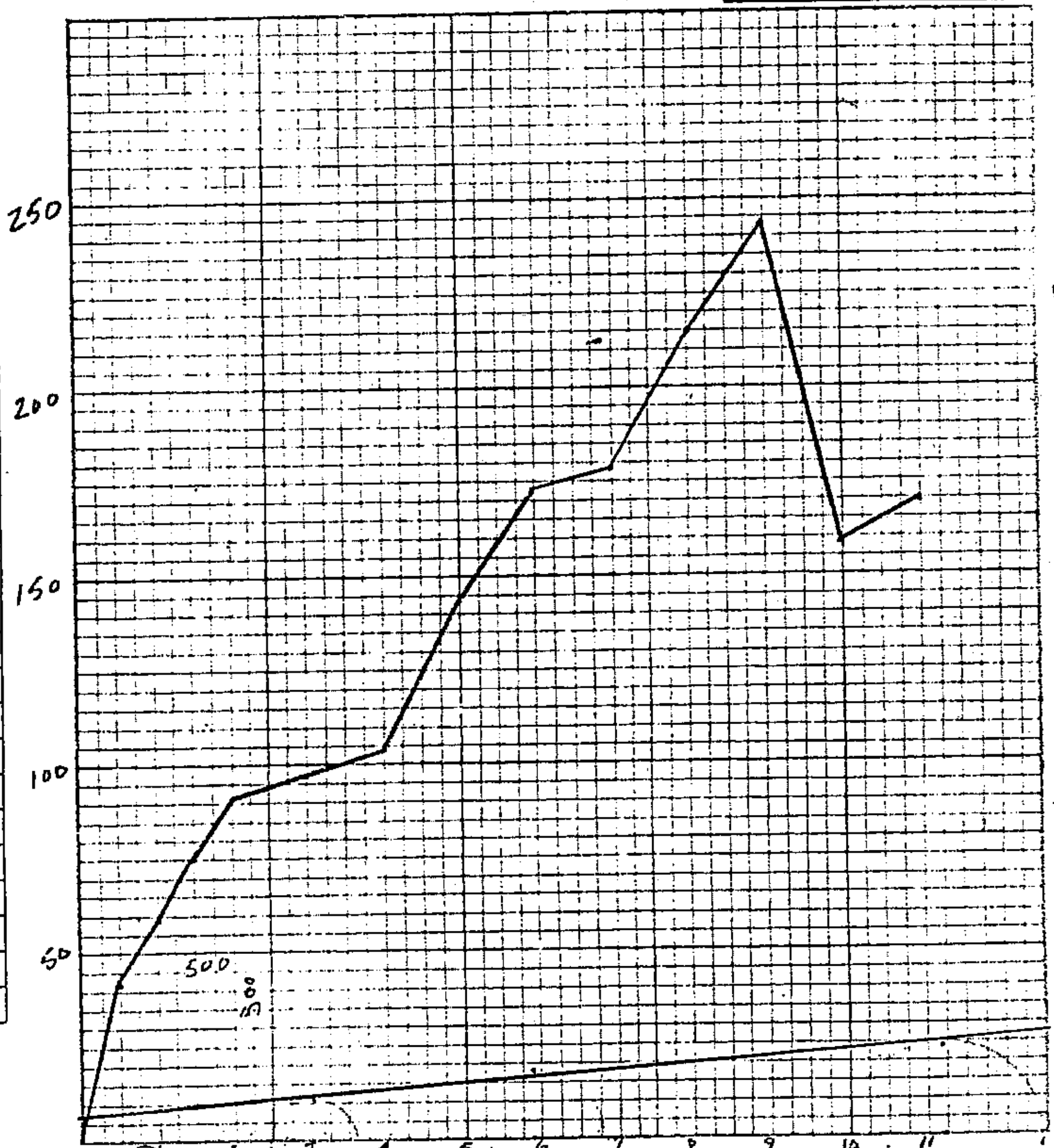
24.0				
25.0				



Instructions
Show Stress-Strain in Red.
Show Mohr's diagram in Black.

Type Machine KW WS ST UW(1) UW(2)

Type Test	Stage ✓	Uncon.	Con. Drained	Con. Undrained	Consol	

Operator TERR.

H-2

TRIAXIAL TEST DATA

2-18-70

Sample No. C-6769-1

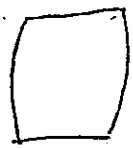
ob No. L-3598 Section RAMP "A" O-XING

Depth 12'2" 12'6" Station A21 +58 Offset 3' RT ℄

aterial Grey Fairly Clean Coarse Sand

Gross Wt.	Tare	Net Wt.	Con No.	Wet Wt.	Dry Wt.	Wt. H_2O	% H_2O	Wet Density	Dry Density
500	145	~~355~~ 361	21	107.96	85.40	22.56	26.1	120.3	95.2

σ_3	$\frac{1}{2}(\sigma_1-\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)-u$
500	530	1030	
1000	650	1650	
2000	790	2790	

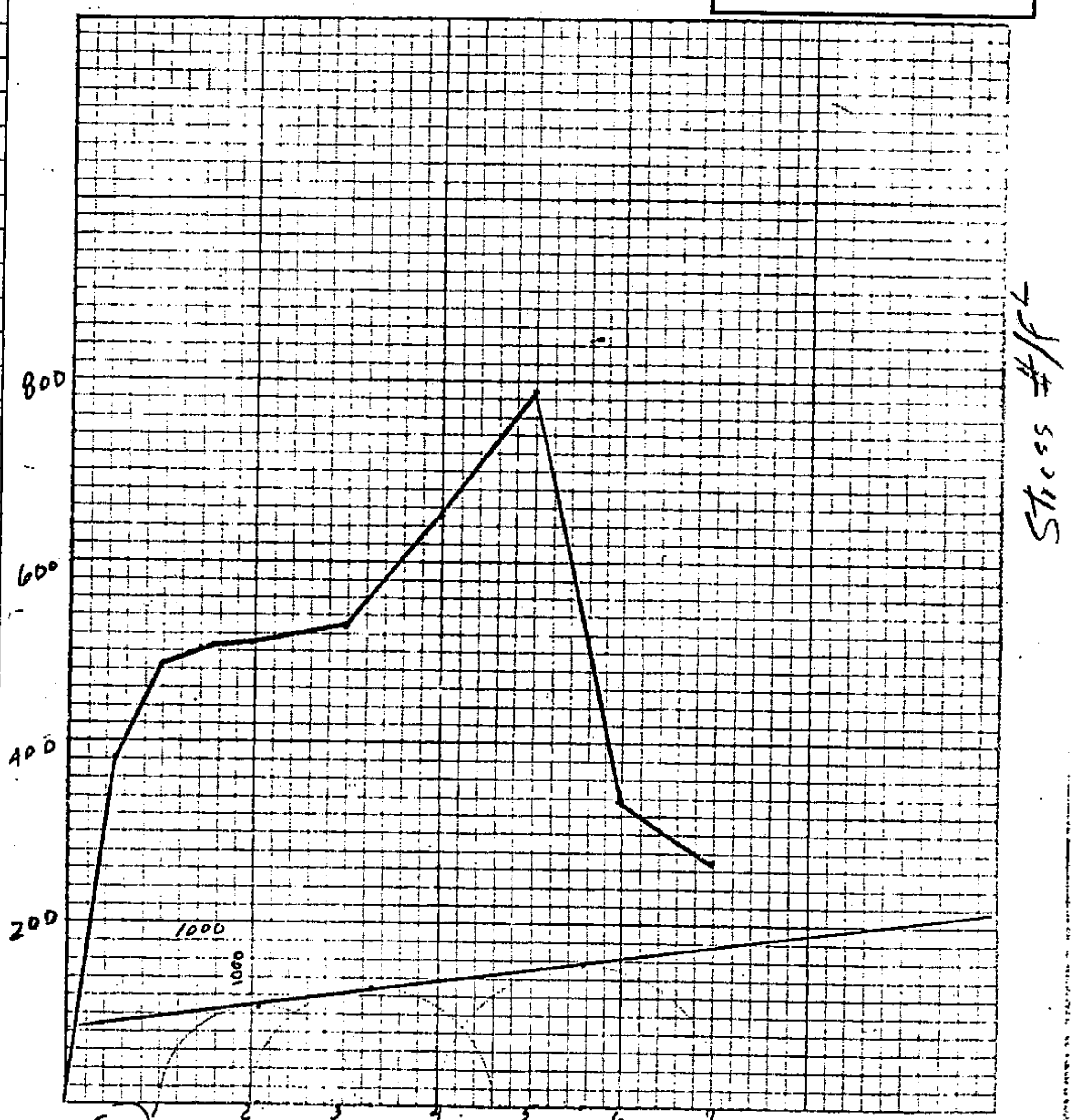


Sample Height = 3.9

C = 400 #/Sq. Ft.

ϕ = 8° Deg.

Strain %	Dial Reading in inch × 10	$\frac{1}{2}(\sigma_1-\sigma_3)$ #/sq. ft.	Strain inch × 10	Press. Gauge #/sq. in.
0			Initial	
0.5	45	380		
1.0	58	485		
1.5	61	505		
2.0	62	510		
3.0	65	530	500	
4.0	80	650	1000	
5.0	98	790	2000	
	42	340	500	
.0	35	270		
8.0				
9.0				
10.0				
1.0				
12.0				
13.0				
4.0				
5.0				
16.0				
7.0				
8.0				
19.0				
0.0				
1.0				
22.0				
3.0				
4.0				
25.0				



pe Machine ✓ KW WS ST UW(1) UW(2)

e Test: Stage | Uncon. | Con. Drained | Con. Undrained | Consol |

Instructions
Show Stress-Strain in Red.
Show Mohr's diagram in Black.

Operator TERRY

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H-2

TRIAXIAL TEST DATA

2-18-70 | Sample No. C-6771-1

Job No. L-3598 | Section RAMP "A" O-xing

Depth 18'4" 19'0" | Station A21+58 | Offset 3' RT ℄

Material Grey Compact Silty Very Fine Sand

Gross Wt.	Tare	Net Wt.	Can No.	Wet Wt.	Dry Wt.	Wt. H_2O	% H_2O	Wet Density	Dry Density
476	144	~~335~~	22	97.55	73.97	23.58	31.9	115.6	87.6
		350							

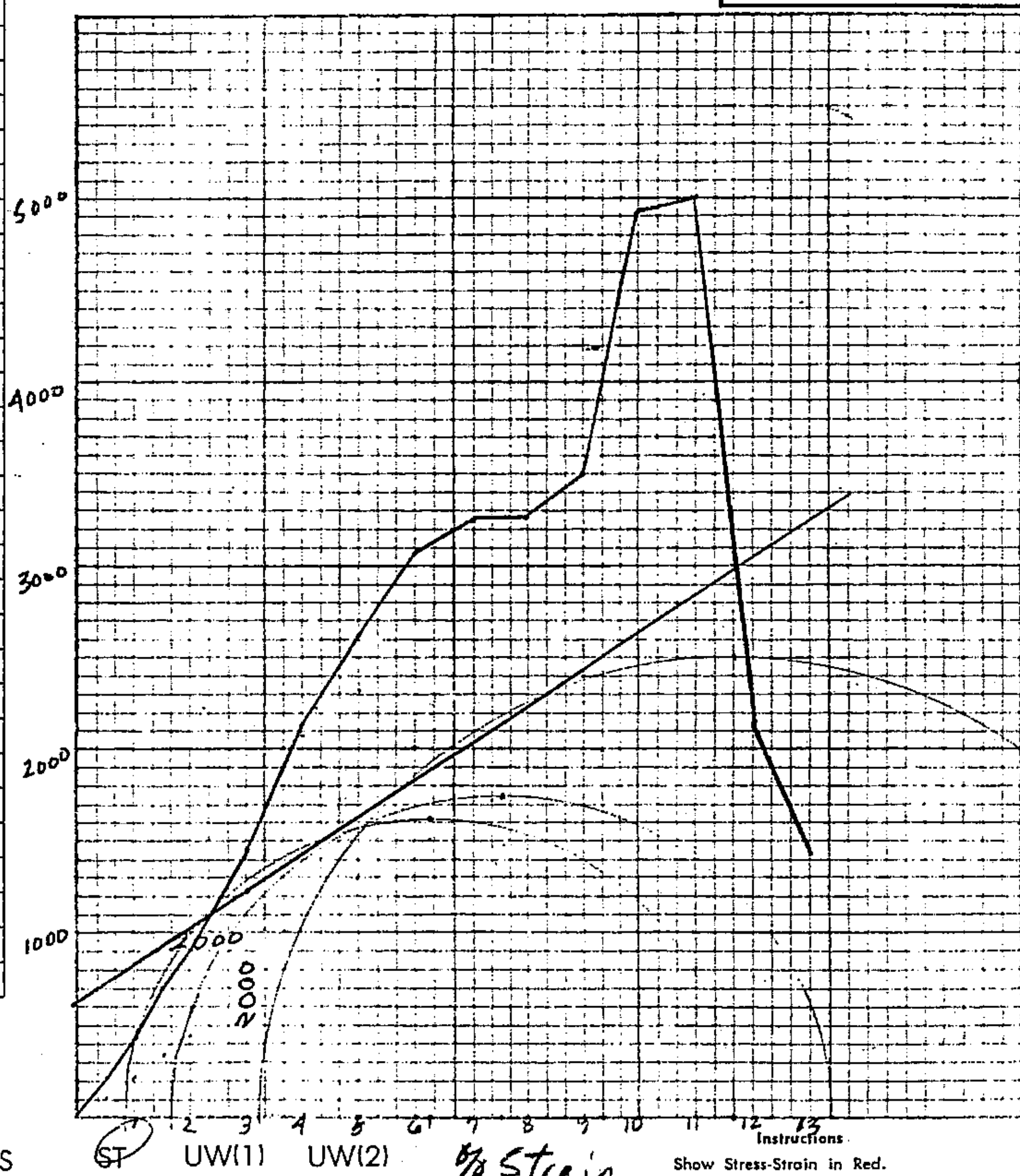
σ_3	$\frac{1}{2}(\sigma_1-\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)-u$
500	3250	3750	
1000	3500	4500	
2000	5000	7000	

Sample Height = 3.8

C = ~~1250~~ 1300 #/Sq. Ft.

ϕ = 31° Deg.

Strain %	Dial Reading in Inch × 10⁻⁴	$\frac{1}{2}(\sigma_1-\sigma_3)$ #/sq. ft.	Strain Inch × 10⁻³	Press. Gauge #/sq. in.
0			Initial	
0.5	25	210		
1.0	50	420		
1.5	84	700		
2.0	110	900		
3.0	180	1460		
4.0	260	2120		
[illegible]	330	2610		
	385	3080		
7.0	416	3250	500	
8.0	420	3260		
9.0	432	3500	1000	
10.0	645	4900		
11.0	668	5000	2000	
12.0	290	2150	500	
13.0	195	1420		
14.0				
15.0				
16.0				
17.0				
18.0				
19.0				
20.0				
21.0				
22.0				
23.0				
24.0				
25.0				



Instructions
Show Stress-Strain in Red.
Show Mohr's diagram in Black.

Type Machine KW WS ST UW(1) UW(2)

Type Test	Stage	Uncon.	Con. Drained	Con. Undrained	Consol	

Operator TERRY

H 2

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3

TRIAXIAL TEST DATA

Date 2-18-70 Sample No. C-6772-1

Job No. L-3598 Section RAMP "A" O'-XING

Depth 21'2" 23'2" Station A 21+58 Offset 3' RT F

Material Grey Silty Sand

Gross Wt.	Tare	Net Wt.	Can No.	Wet Wt.	Dry Wt.	Wt. H_2O	% H_2O	Wet Density	Dry Density
503	148	~~355~~	23	110.88	89.02	21.86	25.7	120.3	95.7
		361							

σ_3	$\frac{1}{2}(\sigma_1-\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)-u$
500	775	1275	
1000	855	1855	
2000	940	2940	

Sample Height = 3.9

C = 650 #/Sq. Ft.

ϕ = 6° Deg.

Strain %	Dial Reading in inch × 10⁻⁴	$\frac{1}{2}(\sigma_1-\sigma_3)$ #/sq. ft.	Strain inch × 10⁻³	Press. Gauge #/sq. in.
0			Initial	
0.5	50	470		
1.0	70	590		
1.5	84	700		
2.0	90	745		
3.0	94	775	500	
4.0	105	855	1000	
0	118	940	2000	
.0	76	605	500	
7.0	79	625		
8.0				
9.0				
10.0				
11.0				
12.0				
13.0				
14.0				
15.0				
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24.0				
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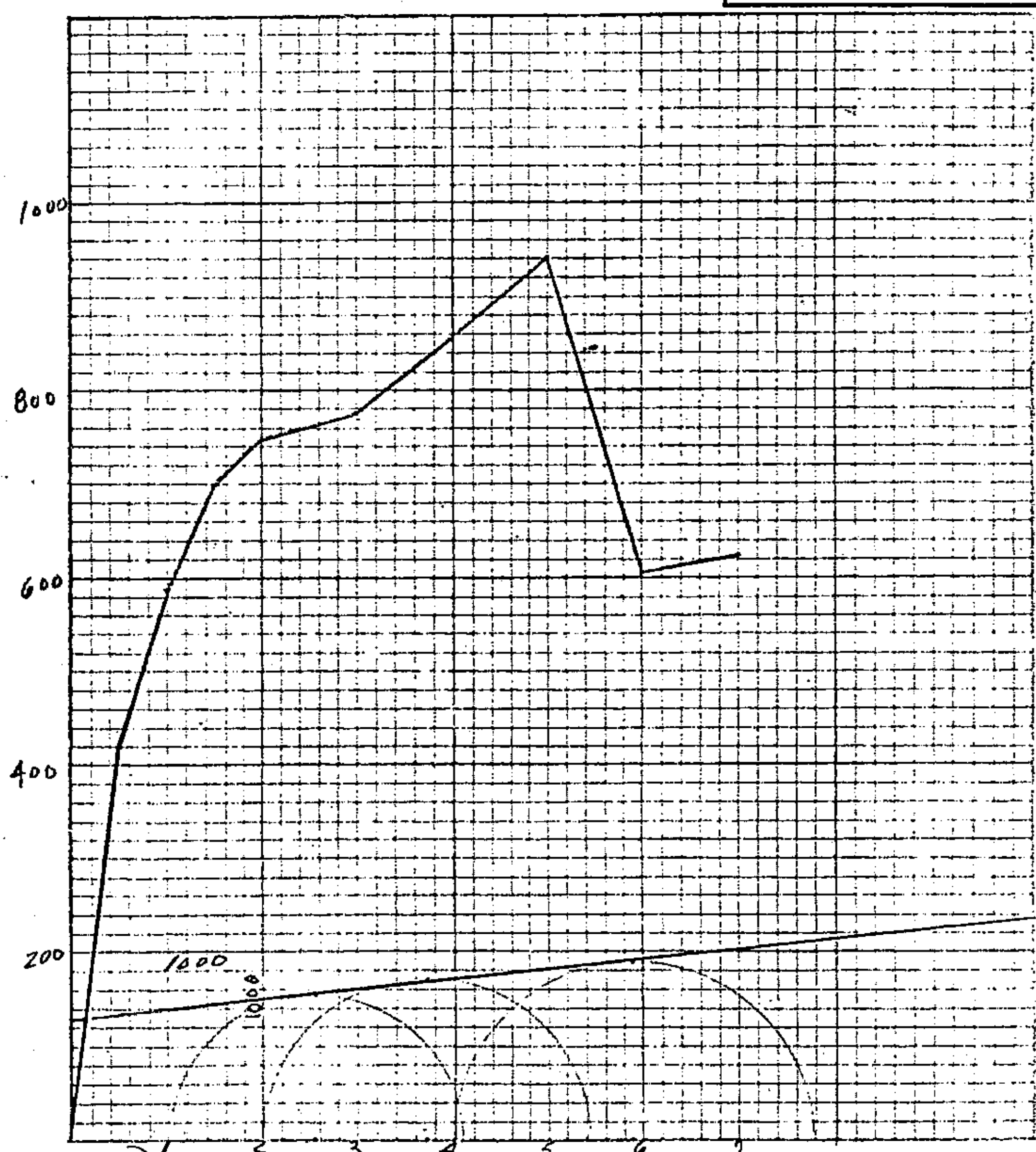


% Strain

Instructions
Show Stress-Strain in Red.
Show Mohr's diagram in Black.

Type Machine KW WS ST UW(1) UW(2)

Type Test	Stage	Uncon.	Con. Drained	Con. Undrained	Consol	

Operator TEKPV

H-2

TRIAXIAL TEST DATA

Date 2-18-70 Sample No. C-6772-1

Job No. L-3598 Section RAMP "A" O'XING

Depth 71'0" 72'0" Station 21+58 Offset 3'RT

Material Grey Sand Silt - Some Org. Mat.

Gross Wt.	Tare	Net Wt.	Can No.	Wet Wt.	Dry Wt.	Wt. H_2O	% H_2O	Wet Density	Dry Density
480	153	327	24	100.55	68.85	31.70	46.0	108.0	74.0

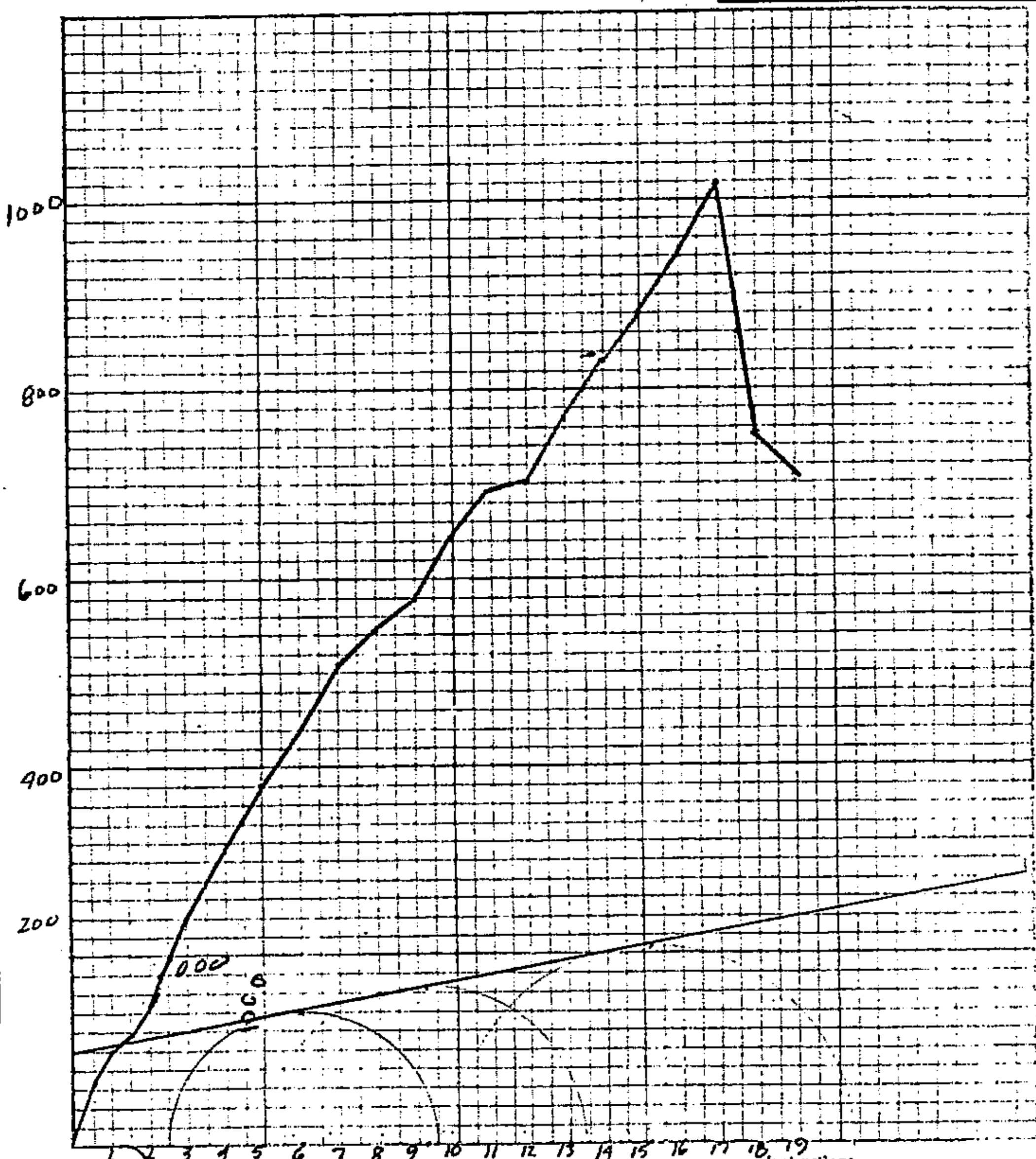
σ_3	$\frac{1}{2}(\sigma_1-\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)-u$
500	700	1200	
1000	830	1830	
2000	1020	3020	

Sample Height = 4.0

C = 500 #/Sq. Ft.

ϕ = 10° Deg.

Strain %	Dial Reading in inch × 10	$\frac{1}{2}(\sigma_1-\sigma_3)$ #/sq. ft.	Strain inch × 10	Press. Gauge #/sq. in.
0			Initial	
0.5	8	67		
1.0	12	100		
1.5	14	115		
2.0	18	146		
3.0	29	234		
4.0	38	310		
0	47	380		
.0	55	440		
7.0	65	510		
8.0	20	595		
9.0	75	580		
10.0	85	645		
11.0	92	695		
12.0	94	700	500	
13.0	107	770		
14.0	115	830	1000	
15.0	126	880		
16.0	135	940		
17.0	147	1020	2000	
18.0	110	755	500	
19.0	105	710		
20.0				
21.0				
22.0				
23.0				
24.0				
25.0				



Instructions
Show Stress-Strain in Red.
Show Mohr's diagram in Black.

Type Machine	KW	WS	ST	UW(1)	UW(2)
Type Test	Stage	Uncon.	Con. Drained	Con. Undrained	Consol

Operator TERRY

1
2
3
4

H-3

TRIAXIAL TEST DATA

Date 2-24-70 — Sample No. C-6818-1

Job No. L-3598 — Section RAMP "A" O-XING

Depth 68'10" 70'2" — Station A 22+33 — Offset 5' LT.

Material Brown Org. Silt with Layer of Silty Sand Top + Bottom

Gross Wt.	Tare	Net Wt.	Can No.	Wet Wt.	Dry Wt.	Wt. H_2O	% H_2O	Wet Density	Dry Density
466	144	322	39	80.55	54.81	25.74	46.9	106.4	72.4

σ_3	$\frac{1}{2}(\sigma_1-\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)-u$
500	1860	2360	
1000	2100	3100	
2000	2350	4350	

γ_{wet} / 1+ω

w = 0.469

106.4 / 1.469

Sample Height = 4.0

C = 1300 #/Sq. Ft.

ϕ = 15° Deg.

Strain %	Dial Reading in inch × 10⁻⁴	$\frac{1}{2}(\sigma_1-\sigma_3)$ #/sq. ft.	Strain inch × 10⁻³	Press. Gauge #/sq. in.
0			Initial	
0.5	8	198		
1.0	15	370		
1.5	22	540		
2.0	30	735		
3.0	49	1180		
4.0	61	1480		
0	69	1640		
0	77	1830		
7.0	80	1860	500	
8.0	91	2090		
9.0	92	2100	1000	
10.0	104	2350		
11.0	104	2320	2000	
12.0	60	1320	500	
13.0	66	1430		
14.0				
15.0				
16.0				
17.0				
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19.0				
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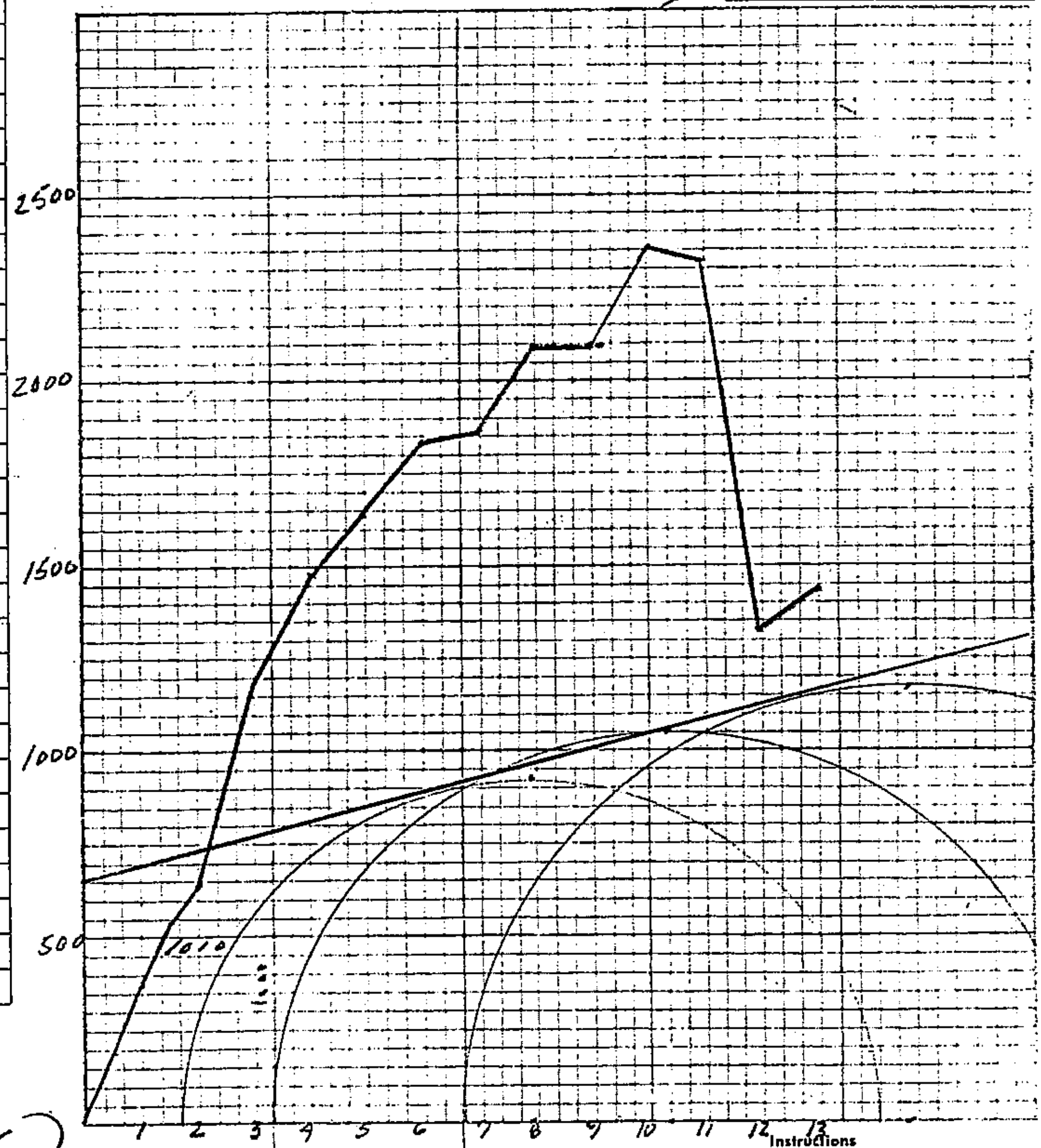
24.0				
25.0				



Instructions
Show Stress-Strain in Red.
Show Mohr's diagram in Black.

Type Machine KW WS ST UW(1) UW(2)

Type Test	Stage	Uncon.	Con. Drained	Con. Undrained	Consol	

Operator Tarry

1
2
3
4

#-4

TRIAXIAL TEST DATA

Date 2-25-70 Sample No. C-6821-4

Job No. L-3598 Section RAMP A O'-XING

Depth 0' 2" 2'2" Station A 23+55 Offset 1'LT

Material Brown Peat

Gross Wt.	Tare	Net Wt.	Can No.	Wet Wt.	Dry Wt.	Wt. H_2O	% H_2O	Wet Density	Dry Density
329	142	187	13	65.80	6.00	59.80	996.7	61.7	5.6

σ_3	$\frac{1}{2}(\sigma_1-\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)-u$
500	97	597	
1000	119	1119	
2000	188	2188	

Strain %	Dial Reading in inch × 10⁻⁴	$\frac{1}{2}(\sigma_1-\sigma_3)$ #/sq. ft.	Strain inch × 10⁻⁴	Press. Gauge #/sq. in.
0			Initial	
0.5	2	50		–
1.0	3	74		
1.5	3	74		
2.0	4	97		
3.0	4	97	500	
4.0	5	119		
[illegible]	5	118	1000	
[illegible]	8	188		
7.0	8	184	2000	
8.0	4	93	500	
9.0	4	92		
10.0				
11.0				
12.0				
13.0				
14.0				
15.0				
16.0				
17.0				
18.0				
19.0				
20.0				
21.0				
22.0				
23.0				
24.0				
25.0				

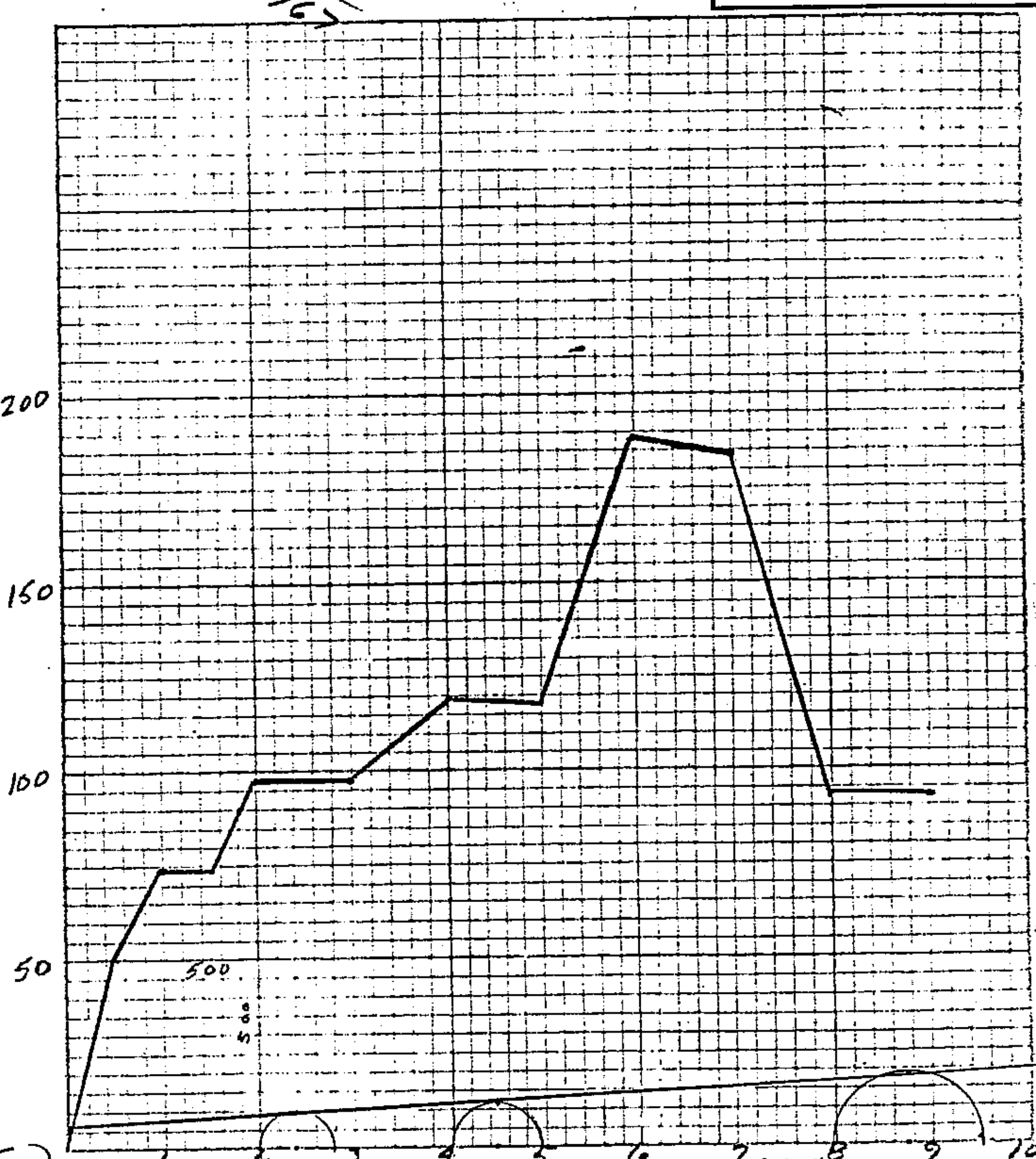
$\frac{61.7}{1+9.967} =$

$\frac{61.7}{10.967}$

Sample Height = 4.0

C = 70 #/Sq. Ft.

ϕ = 3° Deg.



% Strain

Instructions
Show Stress-Strain in Red.
Show Mohr's diagram in Black.

Type Machine KW WS ST UW(1) UW(2)

Type Test	Stage	Uncon.	Con. Drained	Con. Undrained	Consol	

Operator Terry

1
2

11-1

TRIAXIAL TEST DATA

Sample No. C-6823-1

Date 2-25-70
Job No. L-3598
Depth 6'4" 7'0"
Section RAMP "A" O-XING
Station A 23+55
Offset 1' LT.
Material Brown Sandy Silt

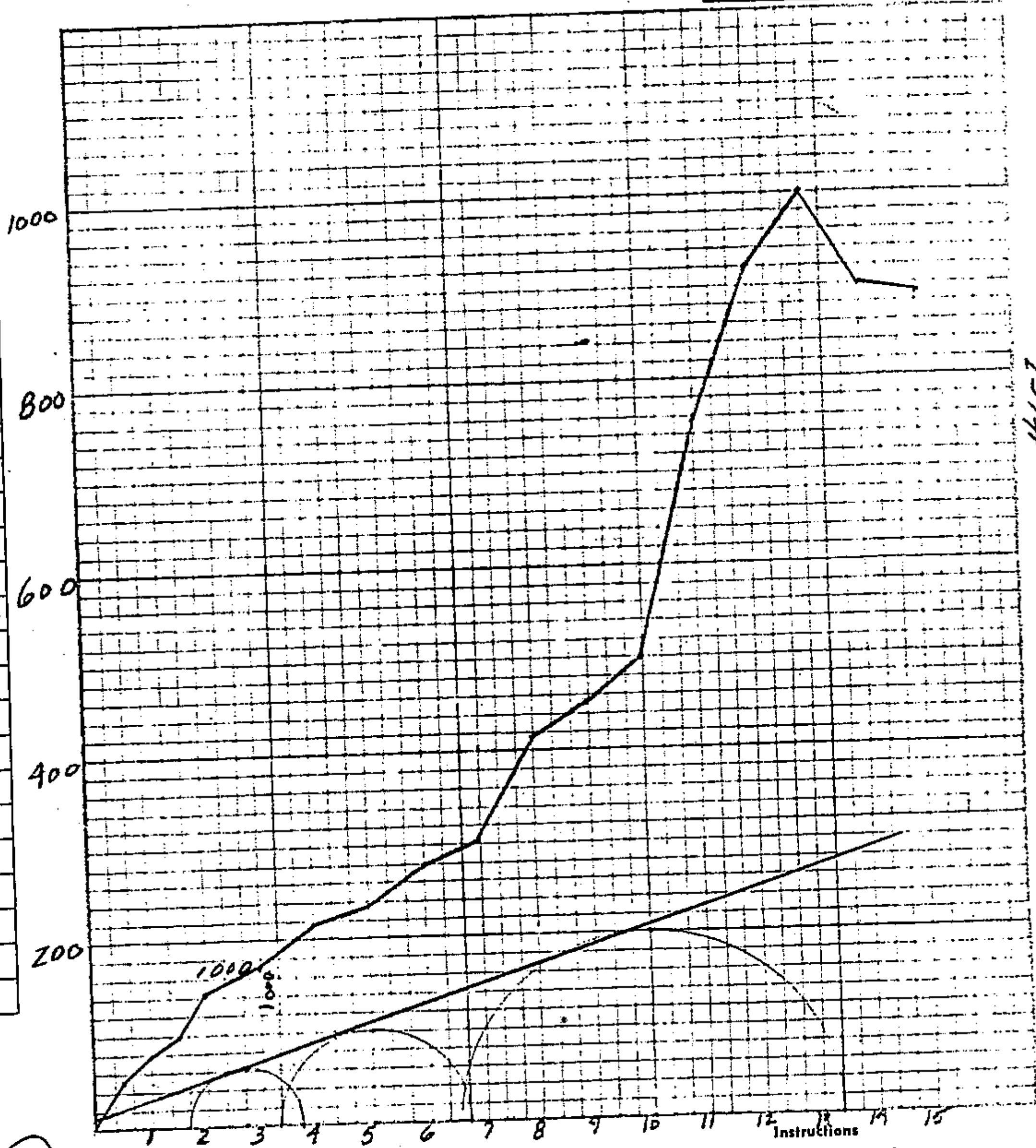
Gross Wt.	Tare	Net Wt.	Can No.	Wet Wt.	Dry Wt.	Wt. H_2O	% H_2O	Wet Density	Dry Density
485	146	339	14	83.83	61.80	22.03	35.6	112.0	82.6

σ_3	$\frac{1}{2}(\sigma_1-\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)-u$
500	305	805	
1000	500	1500	
2000	1000	3000	

Sample Height = 4.0

C = 60 #/Sq. Ft.
ϕ = 19° Deg.

Strain %	Dial Reading in inch × 10	$\frac{1}{2}(\sigma_1-\sigma_3)$ #/sq. ft.	Strain inch × 10	Press. Gauge #/sq. in.
0			Initial	
0.5	2	50		
1.0	3	74		
1.5	4	98		
2.0	6	146		
3.0	7	170		
[illegible]	9	216		
	10	238		
6.0	12	280		
7.0	13	305	500	
8.0	18	415		
9.0	20	455		
10.0	22	500	1000	
11.0	34	755		
12.0	42	920		
13.0	46	1000	2000	
14.0	42	900	500	
15.0	43	890		
16.0				
17.0				
18.0				
19.0				
20.0				
21.0				
22.0				
23.0				
24.0				
25.0				



Instructions
Show Stress-Strain in Red.
Show Mohr's diagram in Black.

Type Machine ✓ KW WS ST UW(1) UW(2)

Type Test	Stage	Uncon.	Con. Drained	Con. Undrained	Consol	

Operator Terry

TRIAXIAL TEST DATA

H.1

Date 2-25-70

Sample No. C-6825-1

Job No. L-3598

Section RAMP "A" O-XING

Depth 11'4" 12'0"

Station A23+55

Offset 1' LT.

Material Grey Fairly Clean Sand

Gross Wt.	Tare	Net Wt.	Can No.	Wet Wt.	Dry Wt.	Wt. H_2O	% H_2O	Wet Density	Dry Density
506	142	364	15	101.60	84.89	16.71	19.7	120.3	100.5

σ_3	$\frac{1}{2}(\sigma_1-\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)-u$
500	510	1010	
1000	1550	2550	
2000	2450	4450	

Sample Height = 4.0

C = 0 #/Sq. Ft.

ϕ = 34° Deg.

Strain %	Dial Reading in inch × 10^{-4}	$\frac{1}{2}(\sigma_1-\sigma_3)$ #/sq. ft.	Strain inch × 10^{-3}	Press. Gauge #/sq. in.
0			Initial	
0.5	12	300		
1.0	17	420		
1.5	19	465		
2.0	20	490		
3.0	21	510	500	
4.0	52	1250		
5.0	65	1550	1000	
6.0	105	2450		
7.0	95	2200	2000	
8.0	57	1310	500	
9.0	58	1320		
10.0				
11.0				
12.0				
13.0				
14.0				
15.0				
16.0				
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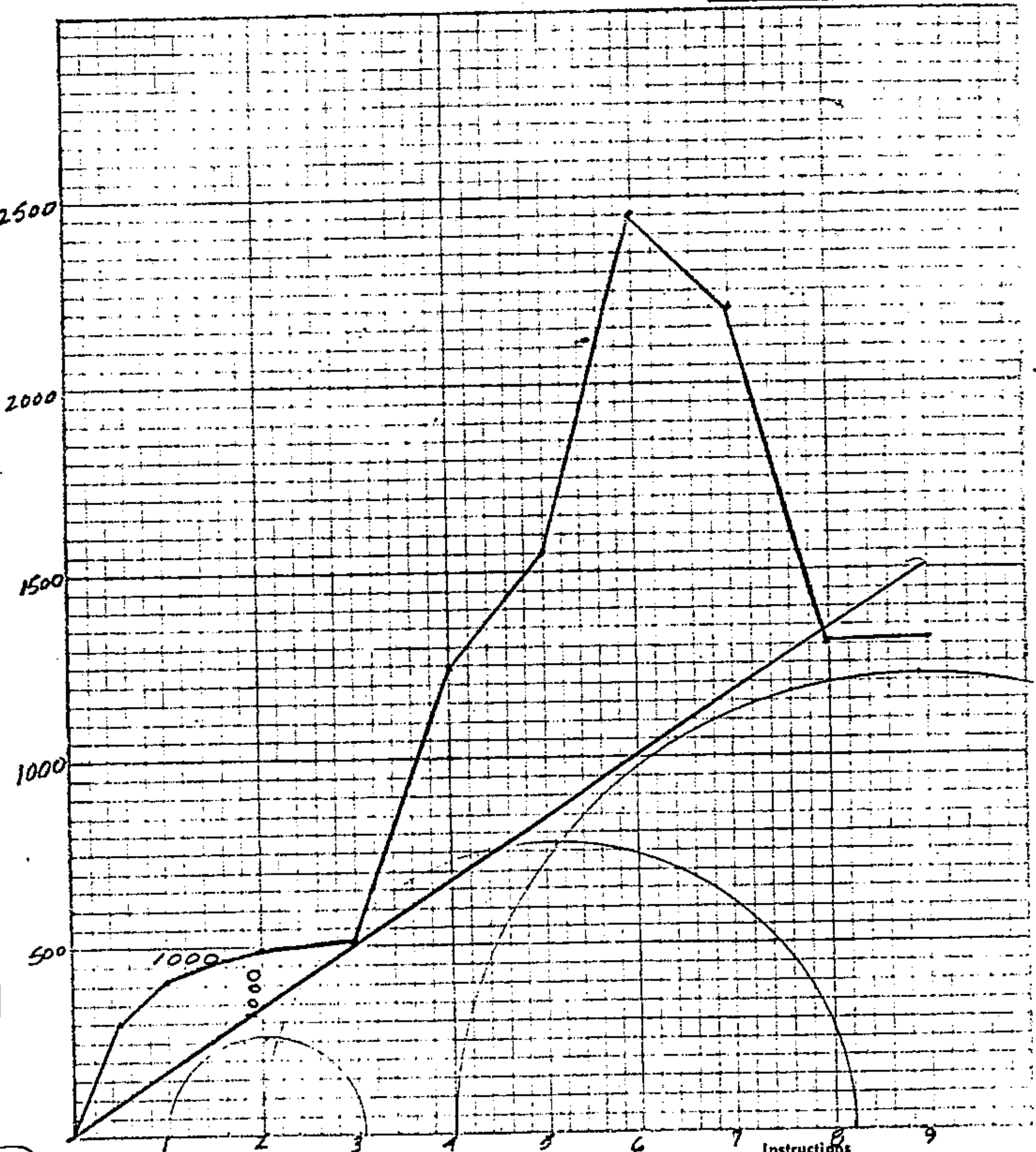
24.0				
25.0				



Instructions
Show Stress-Strain in Red.
Show Mohr's diagram in Black.

Type Machine KW WS ST UW(1) UW(2)

Type Test	Stage	Uncon.	Con. Drained	Con. Undrained	Consol	

Operator T. [illegible]

1
2
3
4

11-2

TRIAXIAL TEST DATA

2-25-70 Sample No. C-6827-1

Job No. L-3598 Section RAMP "A" O-KING

Depth 18'0" 19'2" Station A23+55 Offset 1'LT

Material Grey Silt - Trace Very Fine Sand

Gross Wt.	Tare	Net Wt.	Can No.	Wet Wt.	Dry Wt.	Wt. H_2O	% H_2O	Wet Density	Dry Density
486	153	333	16	81.22	58.30	22.92	39.3	110.0	79.0

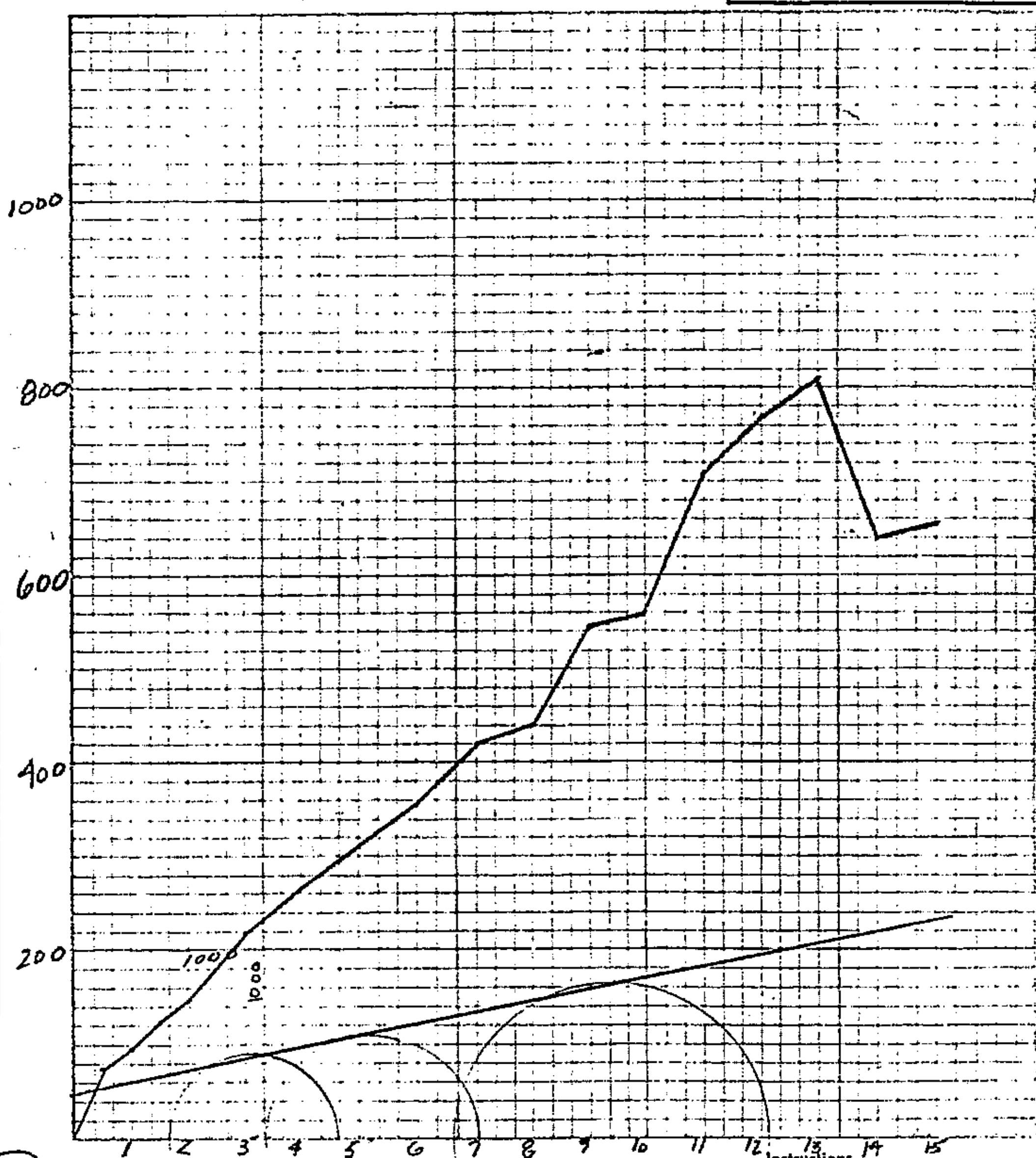
σ_3	$\frac{1}{2}(\sigma_1-\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)-u$
500	440	940	
1000	560	1560	
2000	810	2810	

Sample Height = 4.0

C = 240 #/Sq. Ft.

ϕ = 12° Deg.

Strain %	Dial Reading in inch × 10⁻⁴	$\frac{1}{2}(\sigma_1-\sigma_3)$ #/sq. ft.	Strain inch × 10⁻³	Press. Gauge #/sq. in.
0			Initial	
0.5	3	75		
1.0	4	98		
1.5	5	123		
2.0	6	146		
3.0	9	218		
4.0	11	265		
5.0	13	310		
6.0	15	355		
7.0	18	420		
8.0	19	440	500	
9.0	24	545		
10.0	25	560	1000	
11.0	32	710		
12.0	35	770		
13.0	37	810	2000	
14.0	30	640	500	
15.0	31	655		
16.0				
17.0				
18.0				
19.0				
20.0				
21.0				
22.0				
23.0				
24.0				
25.0				



Instructions
Show Stress-Strain in Red.
Show Mohr's diagram in Black.

Type Machine KW WS ST UW(1) UW(2)

Type Test	Stage	Uncon.	Con. Drained	Con. Undrained	Consol	

Operator TERRY

11/24

H-1

1
2
3
4

TRIAXIAL TEST DATA

2-25-70 Sample No. C-6829-1

Job No. L-3598 Section Ramp "A" O'-xing

Depth 61'4" 62'8" Station A23+55 Offset 1' LT

Material (Saturated) Grey Sandy Silt

Gross Wt.	Tare	Net Wt.	Can No.	Wet Wt.	Dry Wt.	Wt. H_2O	% H_2O	Wet Density	Dry Density
492	147	~~315~~ 302	17	70.19	58.65	19.[illegible]9	33.2	117.0	87.8

σ_3	$\frac{1}{2}(\sigma_1-\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)-u$
500	1910	2410	
1000	2020	3020	
2000	2230	4230	

Sample Height = 3.9

C = 1500 #/Sq. Ft.

ϕ = 10° Deg.

Strain %	Dial Reading in inch × 10^{-4}	$\frac{1}{2}(\sigma_1-\sigma_3)$ #/sq. ft.	Strain inch × 10^{-3}	Press. Gauge #/sq. in.
0			Initial	
0.5	7	174		
1.0	11	270		
1.5	14	345		
2.0	21	510		
3.0	36	870		
4.0	50	1190		
[illegible]	64	1520		
[illegible]	76	1790		
7.0	82	1910	500	
8.0	89	2020	1000	
9.0	98	2230		
10.0	79	1770	2000	
11.0	38	895	500	
12.0	34	750		
13.0				
14.0				
15.0				
16.0				
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19.0				
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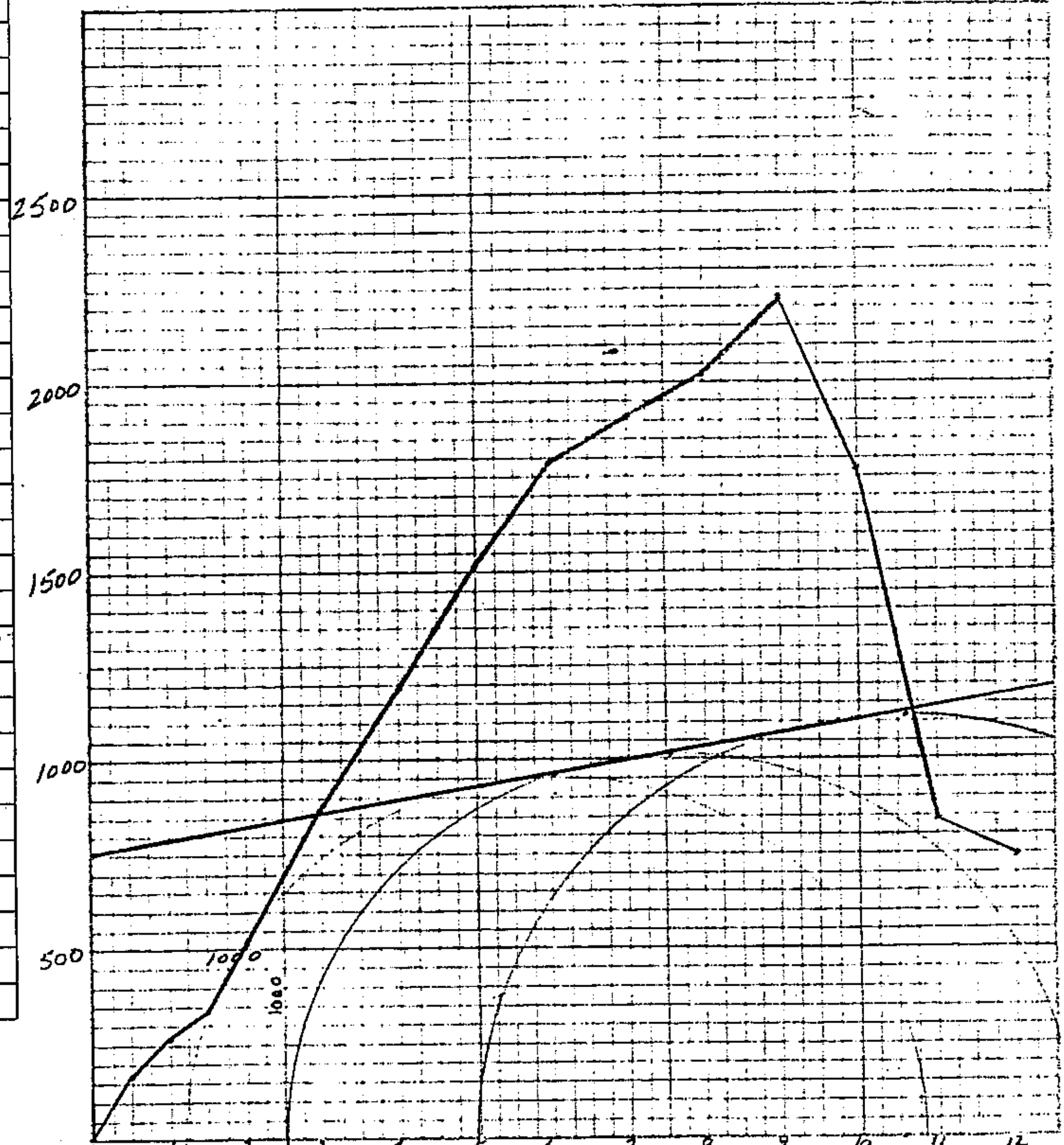
24.0				
25.0				



Instructions
Show Stress-Strain in Red.
Show Mohr's diagram in Black.

Type Machine KW WS ST UW(1) UW(2)

Type Test	Stage	Uncon.	Con. Drained	Con. Undrained	Consol	

Operator TERRY

1
2
3
4

H-d

TRIAXIAL TEST DATA

No. 2-25-70 Sample No. C-6831-1

Job No. L-3598 Section RAMP "A" O'xing

Depth 66'0" 67'8" Station A23+55 Offset 1'LT

Material Gray Silt with Piece of Wood

Gross Wt.	Tare	Net Wt.	Can No.	Wet Wt.	Dry Wt.	Wt. H_2O	% H_2O	Wet Density	Dry Density
445	144	301	18	95.51	68.20	27.31	40.0	99.5	71.1

σ_3	$\frac{1}{2}(\sigma_1-\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)$	$\frac{1}{2}(\sigma_1+\sigma_3)-u$
500	385	885	
1000	560	1560	
2000	765	2765	

Sample Height = 4.0

C = 230 #/Sq. Ft.

ϕ = 12° Deg.

Strain %	Dial Reading in inch × 10	$\frac{1}{2}(\sigma_1-\sigma_3)$ #/sq. ft.	Strain inch × 10	Press. Gauge #/sq. in.
0			Initial	
0.5	4	99		
1.0	7	173		
1.5	9	222		
2.0	11	269		
3.0	14	340		
4.0	16	385	500	
	20	475		
	22	515		
7.0	24	560	1000	
8.0	30	690		
9.0	33	750		
10.0	34	765	2000	
11.0	21	465	500	
12.0	25	550		
13.0				
14.0				
15.0				
16.0				
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19.0				
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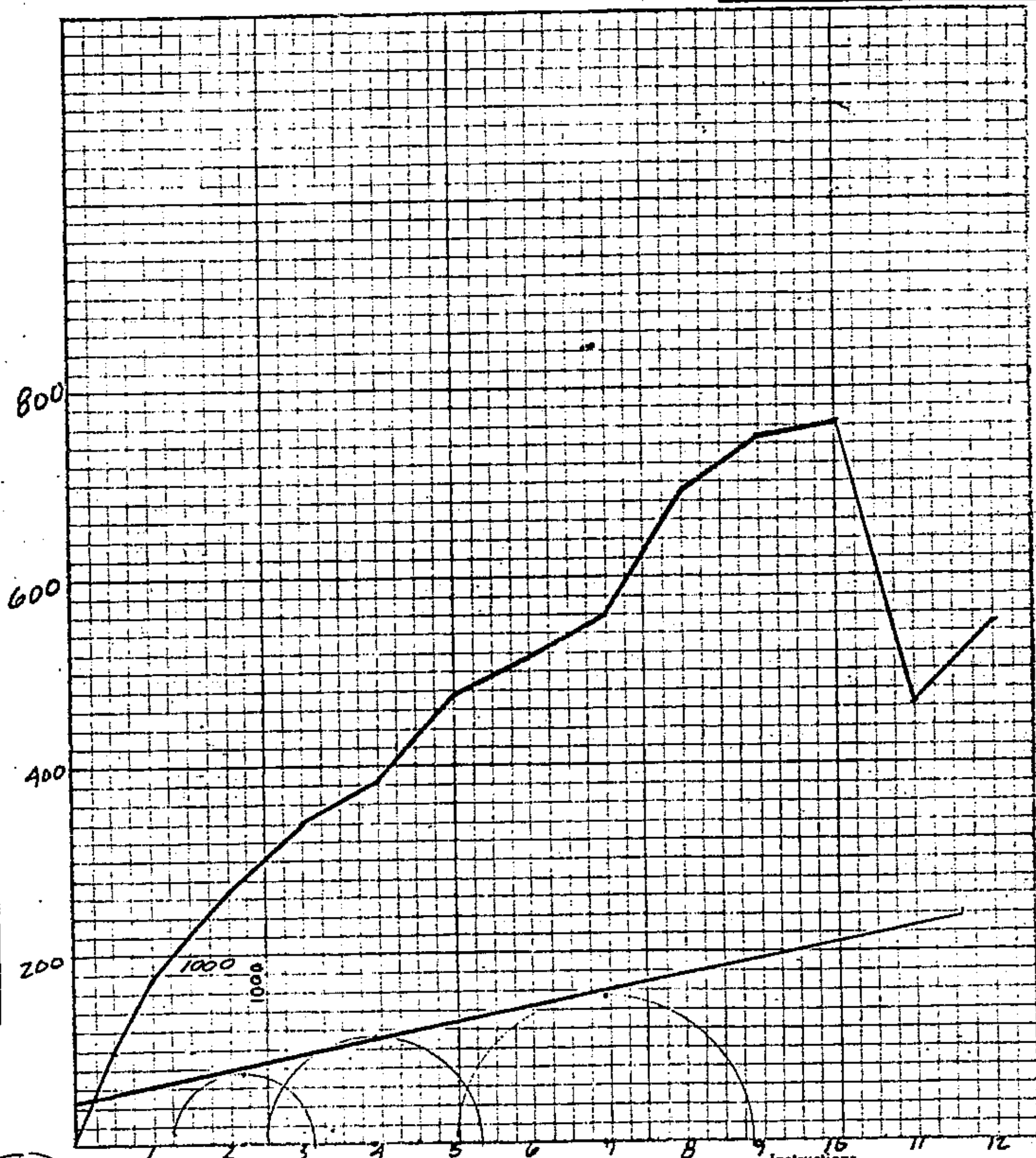
24.0				
25.0				



Instructions
Show Stress-Strain in Red.
Show Mohr's diagram in Black.

Type Machine KW WS ST UW(1) UW(2) % Strain

Type Test	Stage ✓	Uncon.	Con. Drained	Con. Undrained	Consol	

Operator TERRY

Chapter 5 -- Bridge No. 167/112 E Ramp

Foundation design studies carried out for Bridge No. 167/112 E Ramp (E Ramp) included

- determining the axial capacity of driven piles and drilled shafts
- assigning soil properties for use in lateral response analyses of driven piles and drilled shafts, and
- estimating the allowable bearing pressures for the abutment footings.

In view of the potential for liquefaction of sands and silts prevalent in the upper 12 m (40 ft) of soil profile at the bridge site, the possible effects of liquefaction on the axial and lateral capacity of driven piles and drilled shafts, as well as the stability of abutment end slopes, were also evaluated. Methods used during and key results from these foundation capacity and liquefaction analyses are presented in this chapter.

Project Design Considerations

The E Ramp structure is located between 15th Avenue SW and 15th Avenue NW, where SR-167 crosses over SR-18. The general location of the bridge is shown in Figure 1-1. This bridge will be widened on its west side by 5.5 m (18 ft) to provide an HOV lane.

Existing Structure

The E Ramp structure was constructed in the early 1970's from prestressed concrete. It is approximately 105 m (343 ft) in length and 12 m (40 ft) in width. The bridge is supported on four interior piers with each pier consisting of two columns. Columns have an exposed height of approximately 5.5 to 7 m (18 to 23 ft). Interior piers of the bridge are located approximately 18 to 27 m (58 to 90 ft) apart. The ends of the bridge are supported by shallow strip footings located within the abutment fill.

The foundation for each column consists of a pile cap located approximately 1 to 2 m (3 to 7 ft) below the roadway surface. From the original design drawings, it appears that each pile cap is roughly 3 m by 3 m (10 ft by 10 ft) in plan and is located approximately 7.5 m (25 ft) from the adjacent pile cap within the pier. Each pile cap is supported by six driven concrete piles. The estimated average length of the concrete piles, based on the original design drawings, is 12 m (40 ft). This results in the toe of the piles being located at an approximate elevation of 8 m (30 ft). The concrete piles are required in the design drawings for the bridge to have a capacity of 490 kN (55 ton) .

Approach fills for the bridge are approximately 8 m (26 ft) in height. The end of the abutment fill is sloped at 2H:1V (horizontal to vertical). The side slopes on the west side of the approach fill are relatively flat with the top of the area only a few meters below the roadway surface. A 1.5-m (5 ft) wide strip footing is located at each end of the bridge in the approach fill, approximately 3 m (10 ft) below the roadway surface. Design drawings indicate that the allowable bearing pressure on the footing is 290 kPa (3 tsf).

Site Conditions

The site is level except for the grade change to accommodate the approach fills for the bridge. Areas along the west side of the bridge abutments, where widening will occur, are covered with grass. These areas should pose no significant obstructions to construction.

Traffic on the E Ramp bridge and SR-18 are heavy and will present significant construction constraints. Median widths for Piers 3 and 4 are approximately 5 m (16 ft); Piers 2 and 5 are located at the toes of the approach fills.

Subsurface Conditions

Ten test holes have been drilled and sampled for this bridge: three for the original bridge design, three as part of a preliminary widening evaluation, and four as part of this task order. A piezometer was installed in one of the test holes completed for this task order. Locations of the test holes are shown in Figure 5-1 at the end of this chapter. Test hole logs based on past and the most recent explorations are included at the end of this report chapter. Limited numbers of laboratory grain-size tests were also completed as part of this task order. Results of these tests are also included at the end of this chapter.

The geotechnical soil profile for this bridge consists of layered silts, sands, and gravels to the maximum depth of exploration, 44 m (140 ft). Figure 5-2 shows the soil profile that was developed from the test hole logs.

For the purposes of the foundation design studies, six primary soil layers are identified. The characteristics and approximate depths of these layers are summarized as follows, beginning at the ground surface:

- **Layer 1 -- Site Fill:** This material occurs from the ground surface to approximate elevation 17 to 18 m (56 to 60 ft). It appears that approximately 3 m (10 ft) of the site soil were removed during original construction and were replaced with this material. The same material is used for the approach fills to the bridge. Generally the fill is a dense sandy gravel. From location to location and depth to depth, the amount of silt changes. This layer is generally above the water table; blowcounts from the SPT are normally greater than 20.

- **Layer 2 – Sandy Silt Layer:** This layer extends from approximate elevation 17 (56 ft) to approximate elevation 14 m (46 ft) near the southern piers (Piers 2 and 3) and from approximate elevation 18 m (60 ft) to elevation 15 m (49) near the northern piers (Piers 4 and 5). The material is primarily fine silty sand and sandy silt. Blowcounts are often less than 10. It is located below the water table.

- **Layer 3 -- Sand and Gravel Layer:** This layer occurs between approximate elevation 14 m (46 ft) and elevation 1 m (3 ft) near the southern piers and from approximate elevation 15 m (49 ft) to elevation 5 m (17 ft) near the northern piers. The layer consists of a gravelly sand to sandy gravel with some wood debris near the northern pier locations. Blowcounts near the southern piers are often above 25; the blowcounts near the northern piers can be less than 25 with some, near the top and bottom of the layer less than 10.

- **Layer 4 -- Sandy Silt Layer:** This layer occurs between elevation 1 m (3 ft) and elevation –2 m (-7 ft) near the southern piers and between elevation 5 m (17 ft) and elevation 0 m (0 ft) near the northern piers. The layer consists generally of silt and sand with some organics. Some clay is also present near the southern piers. Blowcounts in this layer can be as low as 10.

- **Layer 5 -- Loose Sand Layer:** This layer consists of nearly 15 m (49 ft) of loose silty sand and gravelly sand with some silt. Traces of wood are noted in the test hole logs. Blowcounts range from 5 to 20 or more. Blowcounts in the top 5 m (16 ft) of this layer are often less than 10. Higher blowcounts occur at deeper depths.

- **Layer 6 -- Dense Gravel Layer:** A dense gravel layer is located at elevation -13 to -15 m (-43 to -49 ft). This layer is very consistent in the general area. Blowcounts from the SPT are in excess of 50 blows per 0.3 m (1 ft).

Several important features within the soil profile were identified from the test hole logs. First, low blowcounts occur within Layers 2, 3, 4, and the upper portions of Layer 5. While some of these low blowcounts appear to be caused by heave within the augers during drilling, at least some are thought to represent actual conditions. As discussed subsequently, the low blowcounts in Layers 2 and 3 lead to concerns about the susceptibility of these layers to liquefaction during a design earthquake. The low blowcounts in Layers 4 and especially the top portion of Layer 5 present concerns about the depths at which end bearing can be mobilized in driven piles or drilled shafts.

Another relevant observation during both the present and past exploration programs was the presence of scattered wood fragments and cobbles within the soil profile. A boulder that required blasting with dynamite was encountered at a depth of 33 m (108 ft) in another test hole (H-2).

Groundwater was measured at depths of 1.5 to 3 m (5 to 10 ft) below the ground surface. These depths correspond to approximate elevations of 18 to 20 m (60 to 66 ft). Artesian conditions were also reported at a depth of 14 m (46 ft) during drilling of one test hole (A-4-71). A design groundwater elevation of 20 m (66 ft) was used for static pile and drilled shaft analyses. For liquefaction analyses the groundwater elevation was assumed to be elevation 18 m (60 ft). This lower level for liquefaction analyses represented an expected long-term condition, while the higher elevation was used for pile and drilled shaft design to assure that adequate conservatism was incorporated in design for possible short-term loading conditions.

Engineering Soil Properties

Engineering properties were assigned for each of the primary soil layers to aid in subsequent foundation design computations. Various methods were used to assign these properties, including soil descriptions, blowcounts from the SPTs, and normal engineering judgment. These properties are best-estimated values, rather than lower bound. The fact that the values are best-estimates needs to be recognized as factors of safety are selected for determining the axial capacity of driven piles and drilled shafts. Summaries of these properties are presented in Table 5-1.

Table 5-1. Summary of Estimated Soil Properties at E Ramp

Soil Layer No.	Moist Unit Weight (kN/m^3)	Saturated Unit Weight (kN/m^3)	Friction Angle	
			Southern Piers	Northern Piers
1	19.6	-	33	33
2	-	18.1	29	29
3	-	19.6	33	30
4	-	18.9	30	30
5	-	18.9	30	30
6	-	20.3	35	35

Liquefaction Susceptibility

Liquefaction assessments were conducted using the Seed-Idriss simplified blowcount procedure (Seed and Idriss, 1982) with a peak ground acceleration of 0.35g. As noted in Chapter 3 of this report, the peak firm-ground acceleration for the site is estimated to be 0.29g. This motion is expected to amplify by a factor of approximately 1.2, as the seismic wave propagates through the upper 30 m (100 ft) of soil profile, resulting in a design motion for liquefaction and embankment stability studies of 0.35g.

In the liquefaction assessment blowcounts from both the 1997 and the previous exploration programs were used to estimate the cyclic resistance ratio (CRR) for the soil on a test hole by test hole basis. Blowcounts from all SPTs were adjusted to an energy of 60 percent. An energy ratio of 80 percent was used for the automatic hammer; all other blowcounts were assumed to be measured at an energy of 60 percent. Other CRR correction factors, including those for overburden, fines correction, and earthquake magnitude were consistent with the latest recommendations of Robertson and Wride (1997).

The liquefaction potential, which is equivalent to the factor of safety against the occurrence of liquefaction, at each test hole location was determined by comparing the computed value of CRR to the cyclic stress ratio (CSR) caused by the design earthquake. If the liquefaction potential was 1.1 or lower, the soil was identified as having a high potential for liquefaction during a design earthquake. A check was then made to determine if the material with a high liquefaction potential met the grain size and plasticity criteria identified by Seed and Idriss (1982) as being necessary for a material to be liquefiable. Locations of high liquefaction potential were then plotted on the soil profile for the E Ramp to determine the trend in liquefaction.

Based on the blowcount analyses, it appears that liquefaction could develop between the groundwater location (i.e., elevation 18 m; 60 ft) and elevation 9 m (30 ft) at the E Ramp. This depth range encompasses all of Layer 2 and the upper portion of Layer 3. The potential for liquefaction is not, however, continuous within this elevation range. Rather, many of the blowcounts within the range suggest a low liquefaction potential, with the factors of safety against the occurrence of liquefaction in excess of 2. Individual points of liquefaction were then discounted if adjacent blowcounts were high, under the premise that re-distribution in porewater pressure would moderate the tendency for porewater pressure buildup. Likewise, blowcounts in areas where heave was specifically noted in the test hole log were also discounted.

From these interpretations, it was concluded that the soil between elevation 18 m (60 ft) and 15 m (49 ft) would be the most likely to liquefy on a relatively continuous basis; i.e., the entire layer would be liquefied at one time. Material between elevation 15 m (49 ft) and 9 m (30 ft) would undergo liquefaction on a more localized basis, with some zones of loose sands and silts liquefying but adjacent areas not liquefying.

Methods of Foundation Analyses

Foundation design studies were completed to determine the capacities of shallow and deep foundations that would likely be used during the widening project. The sizes for these foundations were provided by WSDOT's project manager. Approaches for the analyses were discussed with WSDOT prior to and during the analyses to confirm that the methods were generally consistent with WSDOT foundation design requirements.

Driven Pile Design

Axial pile capacities were determined for 460 and 610 mm (18 and 24 in) steel pipe piles. It was assumed that these piles would be driven with a closed end, and filled with concrete after driving. Analyses were conducted for these two pile sizes to determine the (1) axial capacity under static (service load) and seismic conditions, (2) the amount of settlement of a four-pile group under the service loads, and (3) soil parameters for lateral pile capacity determination.

Static Axial Capacity Determination

Both compressive and uplift capacities of the piles were determined. The unified method of design (Fellenius, 1996) was used to estimate compressive and uplift capacities. Coefficients for β and N_t used during these analyses are given in Table 5-2. No limitations were placed on the determination of side and end resistance when computing capacities. In some design methods a critical depth of 10 to 20 pile diameters is imposed, beyond which side friction and end resistance values do not increase (e.g., DM-7, 1982). However, for the depths involved and based on discussions by Fellenius and Altaee (1995), there seems to be considerable question whether the critical depth concept is appropriate.

Table 5-2. Summary of Coefficients for Driven Pile Design at E Ramp

Layer No.	Static Conditions		Seismic Conditions	
.	β	N_t	β	N_t
1	0.35	-	0.35	-
2	0.30	-	0.15	-
3a (> elev. 9) 3b (< elev. 9)	0.45 0.45	55 55	0.15 0.45	- 55
4	0.32	-	0.32	-
5	0.30	35	0.30	35
6	0.45	60	0.45	60

In recognition that the soil layering seems to change between the southern piers (Piers 2 and 3) and the northern piers (Piers 4 and 5), separate analyses were completed to account for somewhat different soil layering and soil properties in each area.

The uplift capacity of the driven piles was assumed to be 80 percent of the friction along the side of the pile in compressive loading. This reduction is consistent with WSDOT's standard practice.

Seismic Axial Capacity Determinations

Procedures used to estimate axial capacity under seismic loading differed from the method for estimating static capacity only in the assigned β value for Layer 2 and part of Layer 3. As discussed above, liquefaction is predicted at various depths in these layers under a design earthquake, the consequence of which will be reduction in the side and end resistance for the pile. It was assumed for the seismic axial capacity determination that liquefaction would occur between approximate elevations 18 and 9 (60 and 30 feet).

Throughout the liquefied zone, a reduced β value was used for side friction. The reduction in side resistance was introduced by using an undrained residual strength ratio (S_r/σ') equal to 0.15. This ratio was selected on the basis of information presented by Dobry and Baziar (1993) and in the draft proceedings from a 1997 National Science Foundation Workshop (NSF, 1997) dealing with the measurement of residual strengths in liquefied soil. A wide range of undrained strength ratios have been suggested for liquefied soil, and some individuals contend that the residual strength is not proportional to the effective overburden pressure. Considering the differences of opinion that currently exist, a check was also performed using the relationship between blowcount and residual strength suggested by Seed and Harder (1990). An undrained strength ratio of 0.15 results in undrained strengths that are not inconsistent with the range determined from the Seed and Harder relationship.

It was further decided that the toe of the pile should be located below the zone with a high risk of liquefaction (i.e., 18 to 9 m; 60 to 30 ft) to minimize the potential for excessive pile settlement during a design seismic event. No adjustments were made for potential buildup in porewater pressure below the liquefied zone. It was assumed that sufficient conservatism had been introduced by establishing the maximum toe elevation below the maximum predicted depth of liquefaction.

This approach to liquefaction was expected to be conservative. The actual effects of the assumption regarding side friction on compressive and uplift capacity are not significant, as the side resistance within this depth interval is relatively small, even under static conditions.

Settlement Estimates for Static Loading

Settlement estimates were made assuming that four piles would be required to support the pile cap for the column. The four-pile configuration was selected primarily to provide increased lateral stiffness, in the event that loss in soil strength occurs in Layer 2 and part of Layer 3 as predicted. It was also assumed that the four piles would be spaced at 2 ½ to 3 diameters.

An equivalent footing approach was taken in estimating settlements. The size of the footing was defined by the perimeter of the pile group. Following discussions with WSDOT engineers, it was decided that the footing would be located at the neutral plane of a single pile, where the neutral plane was defined as the point at which the side friction for the pile equals the service load. A 2V:1H stress distribution was assumed below the footing.

Soil Parameters for Lateral Pile Loading

Procedures used to determine soil parameters for lateral-load analyses generally followed recommendations by Reese and others (e.g., Reese and Wang, 1989a). Modulus of subgrade reaction values were based on information presented in Lam and Martin (1986), which gives modulus of subgrade reaction values as a function of relative density for sands located above and below the water table. These parameters are appropriate for use in the computer programs LPILE and COM624.

For seismic loading the resistance of Layer 2 and Layer 3 was reduced to account for the likelihood of liquefaction under a design earthquake. While liquefaction could occur between elevation 18 and 9 m (60 and 30 ft), it appears that Layer 2, which makes up the upper 3 m (10 ft) of the liquefiable zone, is the most vulnerable. Within this layer a fully liquefied condition was assumed. The average corrected blowcount, $(N_1)_{60}$, for this layer was approximately 12, resulting in a β of 0.15 based on NSF (1997) or a strength of 12 kPa (250 psf) based on the lower bound of the relationship between residual strength and corrected SPT value given by Marcuson et al. (1990). Below approximate elevation 15 the liquefied zone was assigned a friction angle midway between the liquefied and nonliquefied values. The basis for this reduced friction angle was that random locations of liquefaction were predicted potentially between elevations 15 and 9 m (49 and 30 ft). However, other locations within the same depth range did not liquefy. Realizing this, it was reasoned that some loss in lateral support capacity would occur, but more resistance would exist than a fully liquefied state.

Pile-group reduction factors were also defined to account for interaction between piles if the piles are closely spaced, as expected. The reduction factor will depend on the selected spacing ratio (i.e., ratio of center-to-center pile spacing to pile diameter). Significant differences in opinion currently exist within the profession regarding the form and amount of reduction to apply. Based on a recent survey of state departments of transportation (Brown at al., 1998), it was found that reduction factors given in references such as DM-7 (1982), the Canadian Foundation Engineering Manual (1985), and even the Federal Highways Administration (FHWA) Manual *Design and Construction of Driven Piles* (GRL, 1996) are generally viewed as resulting in too much reduction in stiffness. The p-multiplier procedures (e.g., Brown and Bollman, 1996) is currently thought to provide the most realistic representation of group effects, in the absence of dynamic analyses such as given in WSDOT's Design Manual *Foundation Stiffness Under Seismic Loadings* (GeoSpectra, 1997).

Drilled Shaft Design

Axial capacities of three drilled shafts, with diameters of 1.22 m (4 ft), 1.83 m (6 ft), and 2.44 m (8 ft), were determined. It was assumed that a steel casing would be used during installation of these shafts, but that the casing would be removed as the concrete is placed. Analyses were conducted for each shaft diameter to determine (1) the axial capacity under static (service load) and seismic conditions, (2) the possible settlement of the shaft under service loads, and (3) soil parameters for lateral shaft capacity determination.

Static Axial Capacity Determination

The static capacity analyses for the shaft involved determination of side resistance, end bearing, and uplift resistance. Procedures suggested by the FHWA Manual *Drilled Shafts* (Reese and O'Neill, 1988) were generally followed when determining capacity. In this approach the end bearing of the shaft is determined from the product of the uncorrected blowcount (N) times a factor of 57.5 in kPa (or N X 0.6 in tsf), and the side friction for cohesionless soil is based on a computed β value.

Procedures used in the estimate of shaft side resistance deviated from recommendations given in the FHWA manual in one important area. When determining β values, the equation recommended in the FHWA manual was not followed. During a progress review meeting with WSDOT's geotechnical engineers, it was decided that the β values determined from the equation in the FHWA manual were too high in the upper layers of soil and possibly too low in the lower layers. To obtain what were considered to be more representative β values for the soil conditions at the site and the likely construction methods, β was defined as the product of a lateral earth pressure coefficient (k) and the tangent of the interface friction angle.

Shaft capacities for the E Ramp were determined using an Excel spreadsheet tabulation. The values of β and the average N values used for the shaft capacities analyses are summarized in Table 5-3. No adjustments were made to the soil properties for shaft diameters greater than 1300 mm (50 in) based on discussions with WSDOT. As with the driven piles, the uplift capacity was assumed to be 80 percent of the compressive capacity of the shaft.

Table 5-3. Summary of Coefficients for Drilled Shaft Design at E Ramp

Layer No.	Static Conditions				Seismic Conditions			
	Piers 2 & 3		Piers 4 & 5		Piers 2 & 3		Piers 4 & 5	
	β	N	β	N	β	N	β	N
1	0.32	-	0.32	-	0.32	-	0.32	-
2	0.27	-	0.27	-	0.15	-	0.15	-
3a (> elev. 9) 3b (< elev. 9)	0.42 0.42	27 32	0.36 0.36	22 22	0.15 0.42	- 32	0.15 0.36	- 22
4	0.29	8	0.29	8	0.29	8	0.29	8
5a (> elev. -8) 5b (< elev. -8)	0.29 0.29	7 15	0.29 0.29	7 15	0.29 0.29	7 15	0.29 0.29	7 15

Seismic Axial Capacity Determinations

Procedures used to estimate the axial capacity of the shaft under seismic loading differed from the method for estimating static capacity only in the assigned β value for Layer 2 and part of Layer 3. As discussed previously for driven piles, liquefaction is predicted at various depths in these layers under a design earthquake, the consequence of which is reduction is the strength of the layer. It was assumed that the β value would be reduced to 0.15 between elevations 18 and 9 m (60 and 30 ft). The rationale for the selection of β of 0.15 is the same as that given for driven piles. Also similar to the driven pile, it was concluded that the toe of the shaft should be located below the maximum predicted depth of liquefaction.

Settlement Estimates for Static Loading

Settlement estimates were made assuming that a single shaft would support each column. An equivalent footing approach was taken in estimating settlements. The size of the footing was defined by the perimeter of the shaft. This footing was located at the neutral plane of the shaft. As noted before, the neutral plane was defined as the point at which the side friction for the shaft equals the service load. A 2V:1H stress distribution was assumed below the equivalent footing.

Soil Parameter for Lateral Pile Loading

Procedures used to determine soil parameter for lateral-load analyses were the same as those used for driven piles. After discussions with WSDOT's geotechnical engineers, it was decided that no adjustment factors would be given to account for the potential effects of shaft diameters greater than 0.6 m (2 ft), as has recently been suggested in some studies (e.g., ATC, 1996). These parameter are appropriate for use in the computer programs LPILE and COM624.

As with the driven piles, the strength of the soil between elevation 18 and 9 meters (60 and 30 feet) was reduced to account for the likelihood of liquefaction under a design earthquake. While liquefaction could occur throughout the elevation range, the upper 3 m (10 ft) were considered most vulnerable. Within this layer a fully liquefied condition, with a residual strength of 12 kPa (250 psf), was assumed. The lower portion of the range was assigned a friction angle midway between the liquefied and nonliquefied values. The basis for this was the same as discussed previously for driven piles.

Abutment Design

To facilitate the widening, it will be necessary to increase the width of the embankment side slopes by approximately 6 m (20 ft). Abutment footings will also have to be constructed in the approach fill to support the new bridge width. In the case of the abutment fill, the existing slopes are relatively flat; therefore, analyses were only performed to determine the stability of the end slope under seismic loading. For the abutment footings, it was necessary to determine allowable bearing pressures and strain compatible dynamic soil properties for the footing. Procedures used to evaluate these requirements are summarized below.

Abutment Stability

The seismic stability of the end slopes for the abutment fill was determined by conducting stability analyses using the computer program PCSTABL (Siegel, 1974). For these analyses the groundwater was assumed to be located at elevation 18 m (60 ft), which is roughly 3 m (10 ft) below the existing ground surface. The end slope of the embankment was assumed to be 2H:1V. Properties of the embankment material and underlying soils were as defined previously within the discussion of Engineering Soil Properties.

Pseudo-static analyses were conducted with PCSTABL. In this approach the seismic coefficient was varied until a factor of safety approximately equal to 1.0 was defined. Properties were similar to those used for the static analyses, except that Layer 2 was assigned a residual strength equal to 0.15 times the effective overburden pressure (i.e., $S_r = 0.15\sigma'$). The basis for the residual strength determination was presented previously in the discussion for Driven Pile Design. A 3-m (10 ft) layer was used to constrain the depth of the failure surface to the zone where continuous liquefaction was expected.

Estimates of deformation during the seismic event were made using the Newmark simplified method. With this method, an approximate estimate of deformation can be obtained from published relationships between the predicted deformation and the ratio of yield acceleration to peak acceleration.

Allowable Footing Pressures and Dynamic Properties

Each end of the existing bridge is supported on an abutment wall that is supported on a 1.5-m (5 ft) wide strip footing extending across the complete width of the bridge. This footing is located approximately 3 m (10 ft) below the roadway surface. It is anticipated that a similar size footing at the same depth will be used for the widening. Allowable bearing pressures for this footing were determined using conventional bearing capacity theory with allowances for the sloping face of the end abutment. It is understood that the lateral earth pressures for the abutment wall will be based on WSDOT's standard wall design.

Shear modulus, material damping, and Poisson's ratio values were estimated based on recommendations given in the FHWA Manual *Seismic Design of Bridge Foundations* (Lam and Martin, 1986). For these parameter determinations the low-strain shear modulus was selected on the basis of average blowcounts recorded during the SPTs within one footing width below the planned footing elevation. An average shearing strain of 0.02 to 0.2 percent was used to adjust for the level of shearing strain expected during a design event.

Recommendations

This presentation of recommendations is separated into two sections. The first covers the foundation systems, and the second involves construction considerations. While the discussion of construction is limited, recommendations given for design of the foundation systems are dependent on the methods used and observations made during construction. For this reason it is critical that any changes in either site conditions encountered during construction or procedures used during construction be brought to the attention of CH2M HILL in order that the following foundation recommendations can be confirmed for the observed conditions or methods.

Foundations

The methods of analyses described in the preceding section were used to develop geotechnical recommendations for design of driven pile and drilled shaft foundations, abutment footings, and abutment slopes under static and seismic loading conditions. These recommendations are based on best estimates of soil properties. Appropriate consideration should be given to the possibility of different soil properties and soil behavior during selection of factors of safety.

Driven Piles and Drilled Shafts -- Static Loading

The interior columns for the bridge can be supported using either driven piles or drilled shafts.

Axial Capacity: Figures 5-3 through 5-12 present **ultimate** axial capacity versus depth plots for each pile and shaft size. It is emphasize that these capacities are ultimate values; they have not been reduced with factors of safety. The maximum ultimate capacity for driven piles is limited to 4,500 kN (500 tons) to keep the ultimate capacity within the range of applicability of the dynamic formula in Section 6-05 of WSDOT's Standard Specifications.

Allowable capacity values can be determined by applying a factor of safety to the capacities given in the figures. Table 5-4 provides recommended factors of safety for design. As shown in this table, the factor of safety should be selected on the basis of the type of field monitoring that is done before or during pile or shaft installation. It is understood that

WSDOT normally will monitor pile drivability or shaft construction; however, if test piles are driven or a static load test were performed, lower factors of safety would be appropriate.

Table 5-4. Recommended Factors of Safety at E Ramp

Field Confirmation	Driven Piles		Drilled Shafts	
	Compressive Loading	Uplift Loading	Compressive Loading	Uplift Loading
None	3	3	4	4
Standard WSDOT	2.5	1.5	2.5	1.5
Test Piles/PDA	2.25	1.4	-	-
Static Load Test	2.0	1.3	2.0	1.3

Minimum and maximum pile or shaft toe elevations should be used with Figures 5-3 through 5-12 to assure development of the required capacities and to limit settlements. Table 5-5 provides a summary of the minimum and maximum toe elevations for the bearing layers. These elevations were established (1) to avoid locating the toe of the driven pile or drilled shaft in what was thought to be a more compressible material (e.g., Layers 2 and 4), (2) to locate the toe of the shaft or driven pile below the maximum anticipated depth of liquefaction, and (3) in the case of drilled shafts to limit construction to depths to that WSDOT believes can be achieved with out great risk of construction problems.

Layers that should not be used for end bearing due to soil type or liquefaction potential are identified with "NA", meaning not appropriate. It is important to note that the drilled shafts at Piers 4 and 5 should not be located above elevation 0. This elevation requirement is imposed because of the uncertain consistency of Layer 3 at Piers 4 and 5. Blowcounts recorded during the 1997 field exploration program were often low within this depth zone. Although it is possible that the low blowcounts were due primarily to heave during the drilling program, the possibility of very loose materials could not be ruled out. After discussing this issue with WSDOT's geotechnical engineers, it was decided that the toe of the shafts should be located below the zone where low blowcounts were recorded. Should this requirement have significant cost implications, then it may be necessary to conduct further explorations at Piers 4 and 5 to reconcile this issue. If additional explorations are conducted, it would be preferable to conduct these explorations with a cone penetrometer to obtain a continuous determination of soil resistance with depth.

For any layer, a four-pile group or drilled shaft founded between the minimum and maximum toe elevations is expected to develop the capacities given in Figures 5-3 through 5-6 and Figures 5-11 through 5-16 with settlements under service loading of less than 25 mm (1 in).

Table 5-5. Summary of Minimum and Maximum Toe Elevations at E Ramp

Layer Number	Driven Piles				Drilled Shafts			
	Minimum Elev. (m)		Maximum Elev. (m)		Minimum Elev. (m)		Maximum Elev. (m)	
	Piers 2 & 3	Piers 4 & 5	Piers 2 & 3	Piers 4 & 5	Piers 2 & 3	Piers 4 & 5	Piers 2 & 3	Piers 4 & %
1	NA	NA	NA	NA	NA	NA	NA	NA
2	NA	NA	NA	NA	NA	NA	NA	NA
3	9	9	4	6	9	NA	5	NA
4	NA	NA	NA	NA	NA	NA	NA	NA
5	-3	-3	NR*	NR	-3	0	-10	-10

* Not Restricted

Lateral Capacity: Soil properties that should be used for lateral pile capacity analyses under service load conditions are summarized in the LPILE/COM624 forms given in Tables 5-6 and 5-7. The elevations at the top of the first layer should be the bottom of the pile cap for driven piles or 1.5 m (5 ft) below the ground surface at the shaft location.

Table 5-6. LPILE/COM624 Parameters for Service Loading at E Ramp - Piers 2 & 3

Layer No.	Type of Soils	Layer Elevation				Effective Unit Weight		Cohesion		Friction Angle	Coefficient of Subgrade Reaction		Soil Type
		Upper		Lower									
		(m)	(ft.)	(m)	(ft.)	(kN/m^3)	(pcf)	(kPa)	(psf)	(degr.)	(MN/m^3)	(pci)	
1	Sand	-	-	17	56	19.6	125	0	0	33	24	90	4
2	Silt	17	56	14	46	8.3	53	0	0	29	3	10	4
3	Sand w/ gravel	14	46	1	3	9.8	63	0	0	33	16	60	4
4	Silty Sand	1	3	-2	-7	9.1	58	0	0	30	9	35	4
5	Sand w/ silt & gravel	-2	-7	-13	-43	9.1	58	0	0	30	9	35	4
6	Sandy Gravel	-13	-43	-	-	10.5	67	0	0	35	23	85	4

Table 5-7 LPILE/COM624 Parameters for Service Loading at E Ramp - Piers 4 & 5

Layer No.	Type of Soil	Layer Elevation				Effective Unit Weight		Cohesion		Friction Angle	Coefficient of Subgrade Reaction		Soil Type
		Upper		Lower									
		(m)	(ft.)	(m)	(ft.)	(kN/m³)	(pcf)	(kPa)	(psf)	(degr.)	(MN/m³)	(pci)	
1	Sand	-	-	18	56	19.6	125	0	0	33	24	90	4
2	Silt	18	56	15	49	8.3	53	0	0	29	3	10	4
3	Sand w/gravel	15	49	5	17	9.8	63	0	0	33	16	60	4
4	Silty Sand	5	17	0	0	9.1	58	0	0	30	9	35	4
5	Sand w/ silt & gravel	0	0	-14	-49	9.1	58	0	0	30	9	35	4
6	Sandy Gravel	-14	-49	-	-	10.5	67	0	0	35	23	85	4

Group reduction factors should be applied if driven piles have spacing ratios of less than five diameters. The group reduction factors given in the following table were developed from Brown and Bollmann (1996). These values apply to the average stiffness of the pile group.

Table 5-8. Group Efficiency Factors for Driven Piles at E Ramp

Row Spacing	3-Pile Group	4-Pile Group	6-Pile Group
3 diameters	0.75	0.65	0.60
4 diameters	0.90	0.85	0.80
5 diameters	1.0	1.0	0.95

Driven Piles and Drilled Shafts -- Seismic Loading

Figures 5-13 through 5-22 present capacity versus depth plots for each pile and shaft size for seismic loading. These plots can be used with seismic loads to confirm that adequate axial capacity still exists when liquefaction occurs in the upper soil layers. In view of the conservative approach used in considering liquefaction for the axial capacity determinations, a factor of safety of 1.0 and 1.3 should be adequate for driven piles and drilled shafts, respectively, during a seismic event. Realizing the high liquefaction potential in Layers 2 and the upper portions of Layer 3, a minimum toe elevation is established at elevation 9 m (40 ft).

The pile or shaft foundation system could settle during the seismic event. This settlement is expected to result from two sources: (1) the added pile or shaft loads resulting from the inertial response of the structure; and (2) densification of the upper portions of Layer 5.

Settlement from added bridge loads is expected to be small. Settlement from the densification of loose materials in the upper portion of Layer 5 could result in up to 50 mm (2 in) of settlement within Layer 5. Driven piles or drilled shafts founded above Layer 5 could settlement this amount. Similar amounts of settlement would also be expected to occur at the approach fills. If the driven piles or drilled shafts are founded in Layer 6, then settle of the interior piers could occur due to drag loads as loose soils densify; however, this settlement is expected to be small. Settlement would still occur at the approach fills, resulting in differential movements between Pier 1 and Pier 2 and between Pier 3 and Pier 4. The amount of this differential movement could be as much as 50 mm (2 in).

Soil properties that should be used for lateral pile capacity analyses during seismic loading are summarized in Tables 5-9 and 5-10. Group adjustment factors discussed above for static loading should be applied. Inasmuch as the phasing between liquefaction and the maximum inertial forces on the bridge structure is difficult to predict, it is recommended that seismic analyses include lateral capacity evaluations for two cases: (1) a nonliquefied case, which is equivalent to the static case (Tables 5-6 and 5-7), and (2) the seismic case. Design should be based on the more critical of the two.

Table 5-9 LPILE/COM624 Parameters for Seismic Loading at E Ramp - Piers 2 & 3

Layer No.	Type of Soil	Layer Elevation				Effective Unit Weight		Cohesion		Friction Angle	Coefficient of Subgrade Reaction		Soil Type
		Upper		Lower									
		(m)	(ft.)	(m)	(ft.)	(kN/m^3)	(pcf)	(kPa)	(psf)	(degr.)	(MN/m^3)	(pci)	
1a	Sand	-	-	17	56	19.6	125	0	0	33	24	90	4
2	Silt	17	56	14	46	8.3	53	12	250*	-	-	-	1
3a	Sand w/gravel	14	46	9	30	9.8	63	0	0	21	5	18	4
3b	Sand w/gravel	9	30	1	3	9.8	63	0	0	33	16	60	4
4	Silty Sand	1	3	-2	-7	9.1	58	0	0	30	9	35	4
5	Sand w/ silt & gravel	-2	-7	-13	-43	9.1	58	0	0	30	9	35	4
6	Sandy Gravel	-13	-42	-	-	10.5	67	0	0	35	23	85	4

* Note: For Layer 2, assume $\varepsilon_{50} = 0.02$ mm/mm

Table 5-10. LPILE/COM624 Parameters for Seismic Loading at E Ramp - Piers 4 & 5

Layer No.	Type of Soil	Layer Elevation				Effective Unit Weight		Cohesion		Friction Angle	Coefficient of Subgrade Reaction		Soil Type
		Upper		Lower									
		(m)	(ft.)	(m)	(ft.)	(kN/m^3)	(pcf)	(kPa)	(psf)	(degr.)	(MN/m^3)	(pci)	
1a	Sand	-	-	18	60	19.6	125	0	0	33	24	90	4
2	Silt	18	60	15	49	8.3	53	12	250*	-	-	-	1
3a	Sand w/gravel	15	49	9	30	9.8	63	0	0	21	5	18	4
3b	Sand w/gravel	9	30	5	17	9.8	63	0	0	33	16	60	4
4	Silty Sand	5	17	0	0	9.1	58	0	0	30	9	35	4
5	Sand w/ silt & gravel	0	0	-14	-49	9.1	58	0	0	30	9	35	4
6	Sandy Gravel	-14	-49	-	-	10.5	67	0	0	35	23	85	4

* Note: For Layer 2, assume ε_{50} = 0.02 mm/mm

Abutment Footings

The abutment footing should be designed for an allowable bearing pressure of 290 kPa (3 tsf). With this loading the settlements are expected to be less than 25 mm (1 in). Roughly half of the settlement is expected to occur during construction of the footing and abutment wall. For seismic loading (i.e., Load Case 7) the allowable pressure on the abutment footing can be increased by a factor of 2.

Shear modulus, material damping, and Poisson's ratio values given in Table 5-11 are recommended for determining stiffness values for seismic design. These values were developed using a shear wave velocity of 250 mps (820 fps), which results in a low-strain shear modulus of approximately 120 MPa (2,500 ksf).

Table 5-11. Dynamic Soil Properties for Abutment Footing at E Ramp

Mode of Vibration	Shearing Strain = 0.02%	Shearing Strain = 0.2%
Shear Modulus	80 MPa (1,700 ksf)	30 MPa (630 ksf)
Material Damping	5%	12%
Poisson's Ratio	0.35	0.35

In the event that future design studies determine that strip footings cannot be used, because of the available room or for whatever other reason, it would be possible to use drilled shafts or driven piles to support the abutment wall. Axial and lateral capacity information presented in this chapter for the closest pier can be used for drilled shaft and driven pile designs at the abutment should a spread footing not be feasible.

Embankment Slopes

The side slopes in the widened area should not exceed 2.5H:1V, which is the maximum existing side slope. End slopes should not exceed 2H:1V, which is also the existing slope steepness. For these slope angles the factor of safety for static loading will be greater than 1.5.

During a design seismic event, deformations of the end slopes and side slopes could occur. The amount of deformation is estimated to be less than 0.3 m (1 foot). Deformations at the end slopes could impose loads on the foundations for the columns. These loads would be imposed on the existing foundations, as well as the foundations for the widening project. In the event that at some future date a seismic retrofit is performed for the widened bridge, the retrofit should consider the potential effects of these additional loads on the foundation system. These effects could be evaluated by conducting lateral analyses of pile or shaft foundations with an imposed load from the moving soil. If the level of deformations cannot be tolerated, various ground improvement methods could be considered as part of the overall retrofit program.

Construction

Construction of the foundations for the widening project requires consideration of a number of issues related to both quality control and difficulties associated with construction. A number of these issue specific to this project site are summarized below. In most cases the contractor should be made aware of these issues or requirements at the time of bidding.

Driven Piles

The primary issues and requirements associated with the use of driven piles are as follows:

- The potential for wood and cobbles exists throughout the soil profile, and particularly in Layers 3 to 6. While these conditions were not widespread, sufficient cases were noted during the drilling of test holes to warrant consideration during the contracting of pile installation. Pile driving contractors should be advised of this possibility within the special provisions.

- In recognition of the uncertainties of axial pile capacity between the southern and northern piers, test piles should be installed prior to establishing pile order lengths. These test piles should be of the same size and should be driven with the same equipment as will be used during construction.

Table 5-10. Recommended Test Pile Program at E Ramp

Bridge	Pier Number	Number of Tests
E Ramp	2	1
	3	1
	4	1
	5	1

- Groundwater could be located within 1.5 to 3 m (5 to 10 ft) of the ground surface. Depending on the location of the bottom of the pile cap, excavations below the ground water elevation could be required. The permeability of Layer 1, in which the pile cap would likely be located, is expected to be high. With this high permeability, it would be essential for the contractor to have identified procedures for handling excess water in the excavation. If winter construction is anticipated, seals may be required to control water. If summer construction occurs, dewatering systems may be sufficient to control water.
- Site access will be very restricted for this bridge. It will likely require lane closures and, possibly, rerouting of traffic.

Drilled Shafts

The primary construction issues and requirements for drilled shaft will be as follows:

- The water table is very high for the site, and soils are primarily cohesionless. This will necessitate the use of steel casing from the ground surface to the maximum depth of construction. It is critical that the casing be removed during placement of concrete, as friction values used for shaft capacity design are based on a soil-concrete interface and not a soil-steel interface. If the casing cannot be removed, shaft side resistance could decrease by as much as 50 percent.
- Shaft lengths could be up to 30 m (100 ft) in length to meet lateral fixity requirements during seismic events. For these lengths quality control during placement of concrete will be critical. Realizing the potential consequences of poor quality control, WSDOT should plan to conduct sonic crosshole logging in each shaft following construction.
- Access will be a significant construction consideration for each pier location.

Abutment Footing

The primary issues related to the construction of the abutment footing are as follows:

- It will likely be necessary to use sheet piling to support the existing abutment fill during excavation for and construction of the new footing. The depth of excavation for the footing will be 3 to 4 m (10 to 13 ft), if the footing is similar in size to the existing footing (i.e., 1.5 m - 5 ft). However, if a wider footing is needed to needed to meet slope-setback requirements, deeper excavations may be required.
- In the event that the new footing is located below the existing footing, special care will be required to avoid loss of footing support for the existing footing during construction. Sheet piling or other support methods are available to provide this support. However, it should be made clear in the special provisions that support of the existing footing must be maintained. It would be desirable to survey the vertical elevations of the abutment wall before construction to be able to quantify any movement that does occur.
- Considering the potential for layers of siltier materials at the base of the planned footing excavation, the footing excavation should be carried to at least 0.3 m (1 ft) below the planned base of the footing. Crushed ballast should be compacted to the base of the footing to assure good drainage and high base friction.

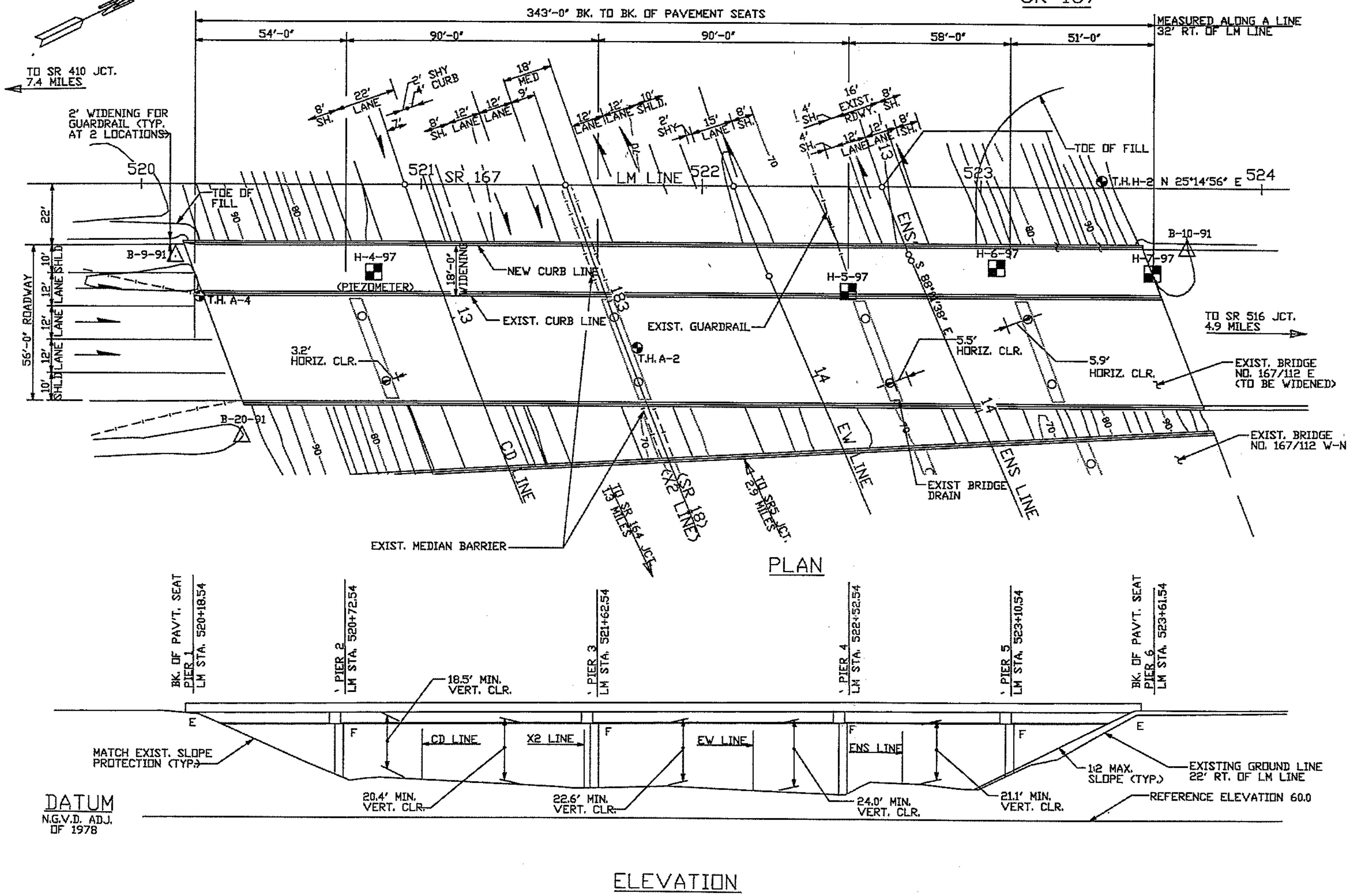
Abutment Slopes

The primary construction issues and requirements related to the abutment slopes are as follows:

- The new side slope fill should be keyed into the existing fill by cutting benches into the existing embankment, as specified in WSDOT's standard specifications.
- Concrete slope protection matching the existing slope protection should be used to prevent ravelling of embankment materials beneath the bridge.

116184.G4 - E PLAN - 01/09/98 112ew1.dwg

SEC. 14, T.21N., R.4E., W.M.
CITY OF AUBURN



NOTES

1. DRAWING ADAPTED FROM PRELIMINARY BRIDGE PLAN DATED OCTOBER 1997. FINAL PLANS MAY VARY.
2. 1 FOOT = 0.305 METERS

SYMBOLS

- EXISTING WSDOT TEST HOLE
- 1997 WSDOT TEST HOLE
- 1991 TERRA TEST HOLE

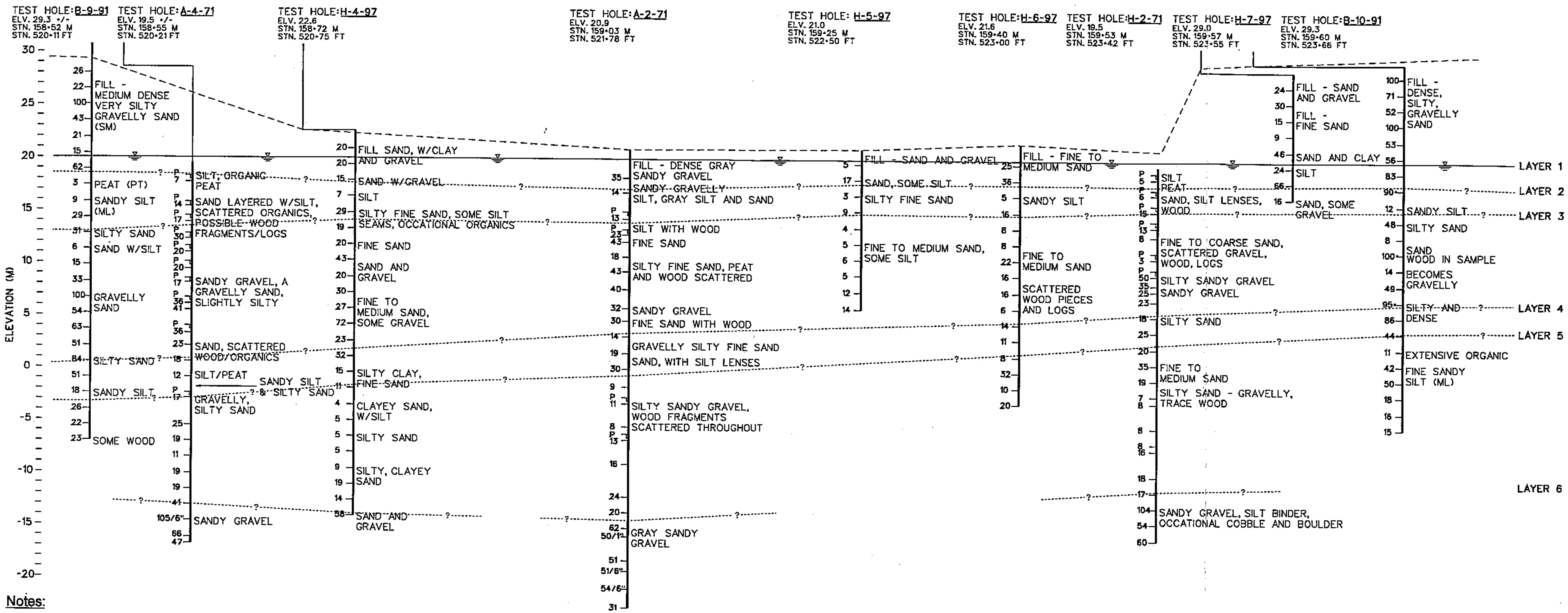
0 40 80
SCALE

Washington State Department of Transportation

CH2MHILL

FIGURE 5-1
E-RAMP WIDENING
TEST HOLE LOCATIONS

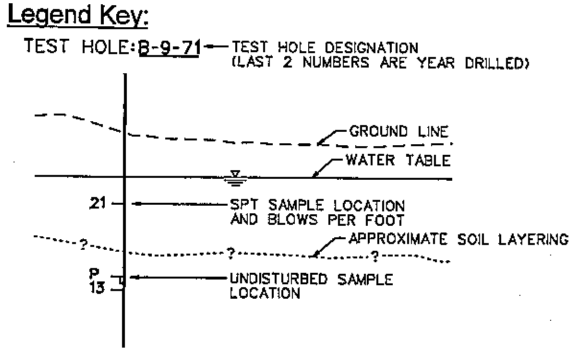
C.S. 1765, PROJECT NO. OL-2305, NORTHWEST REGION, 15TH ST. SW TO 15TH ST. NW, HOV LANE-STAGE 3, SR 167, BR. NO. 167/112 E RAMP WIDEN.



Notes:

1. Soil layering is based on interpretations from soil test hole logs and engineering judgment. Actual conditions within and between test holes could differ from those indicated.
2. Water table elevation based on maximum estimated conditions. Actual elevation could be as much as 3 meters below identified elevation.

Legend Key:



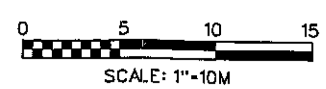


Figure 5-2

Soil Profile For Bridge No. 167/112 E Ramp Geotechnical Report

SR-167, OL-2305
15th Avenue, SW To 15th Avenue NW
HOV Widening Project

CH2MHILL

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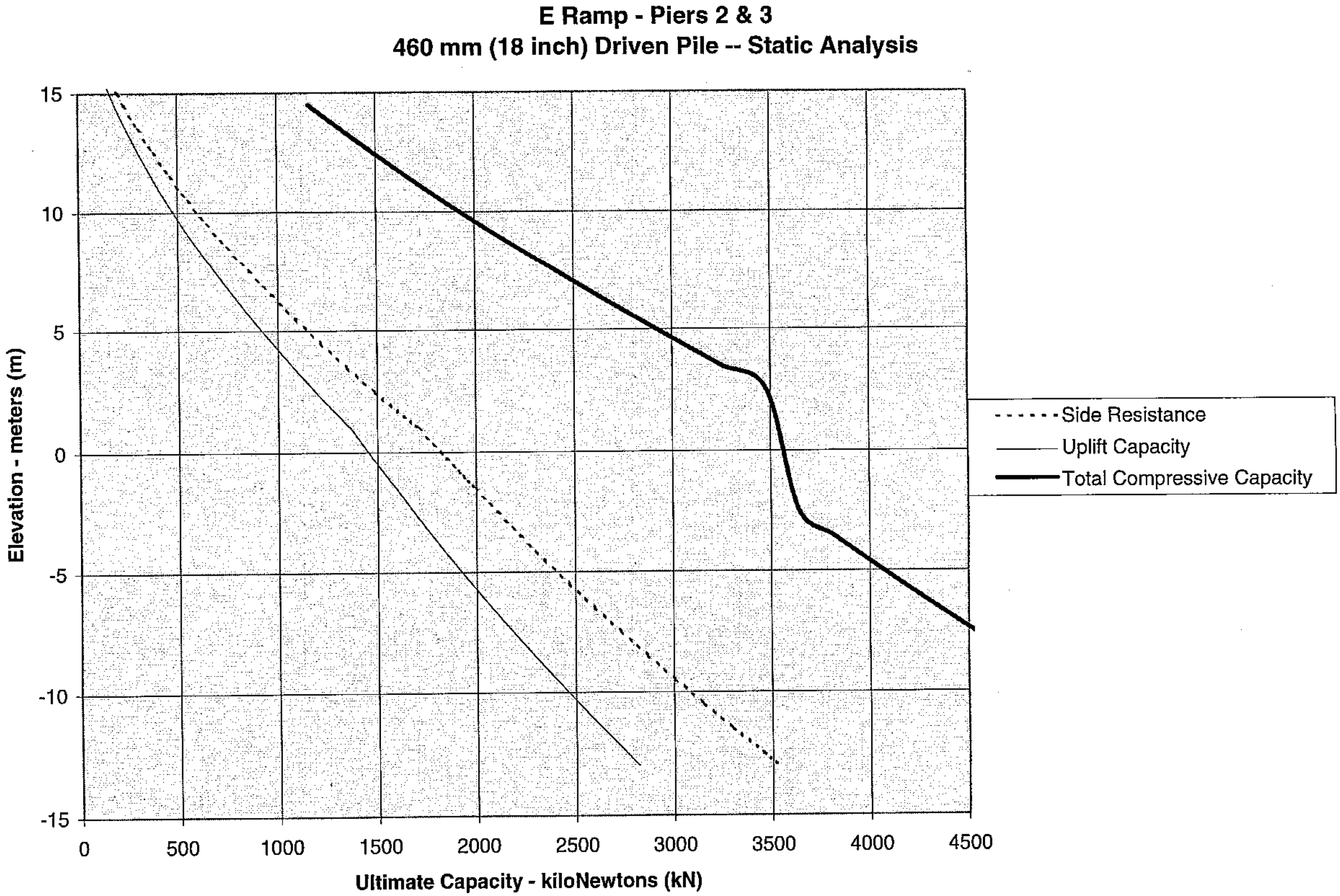


Figure 5-3. Ultimate Pile Capacity for 460 mm (18 inch) Driven, Closed-End Steel Piles for Piers 2 & 3 at E Ramp – Static Analysis

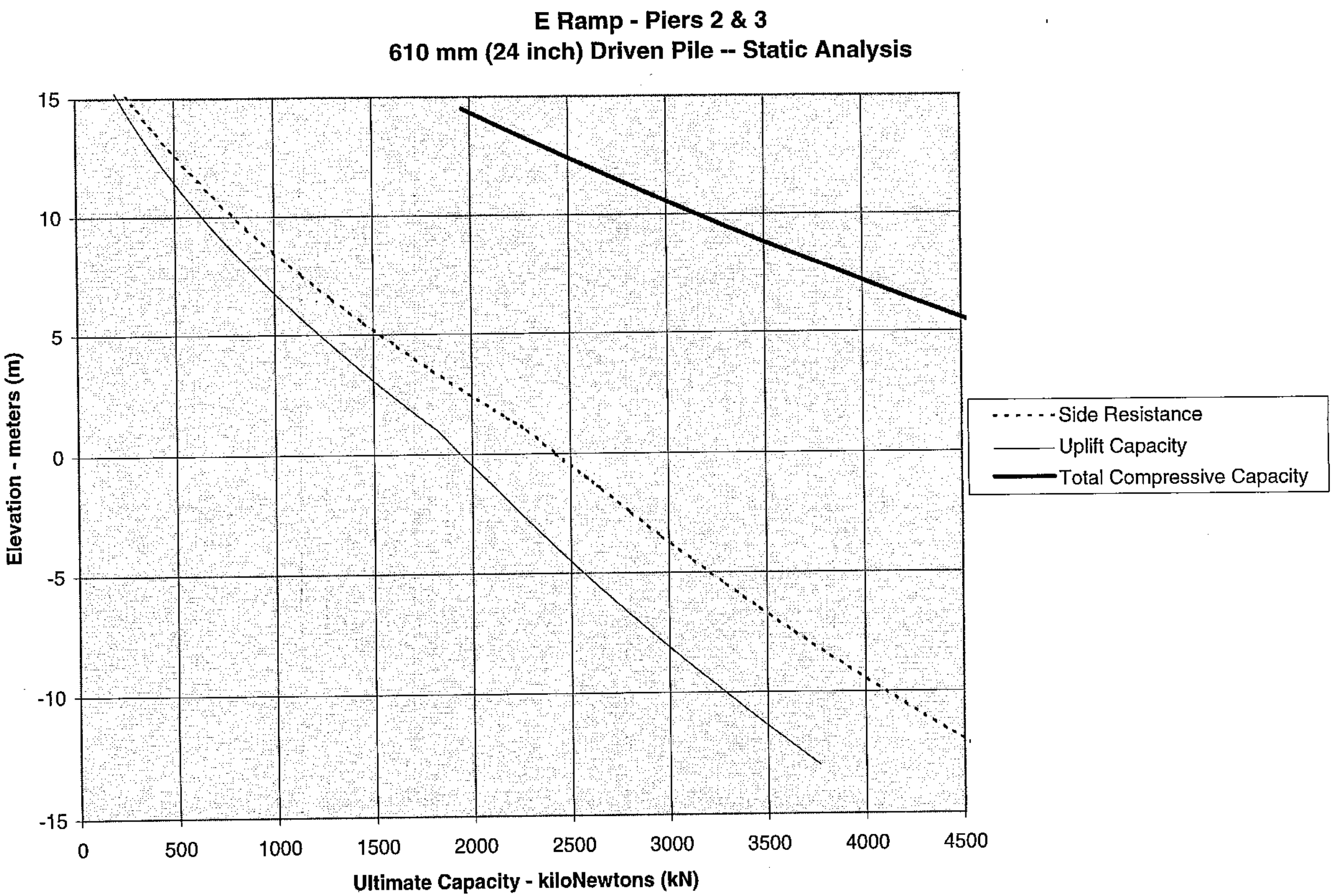


Figure 5-4. Ultimate Pile Capacity for 610 mm (24 inch) Driven, Closed-End Steel Piles for Piers 2 & 3 at E Ramp – Static Analysis

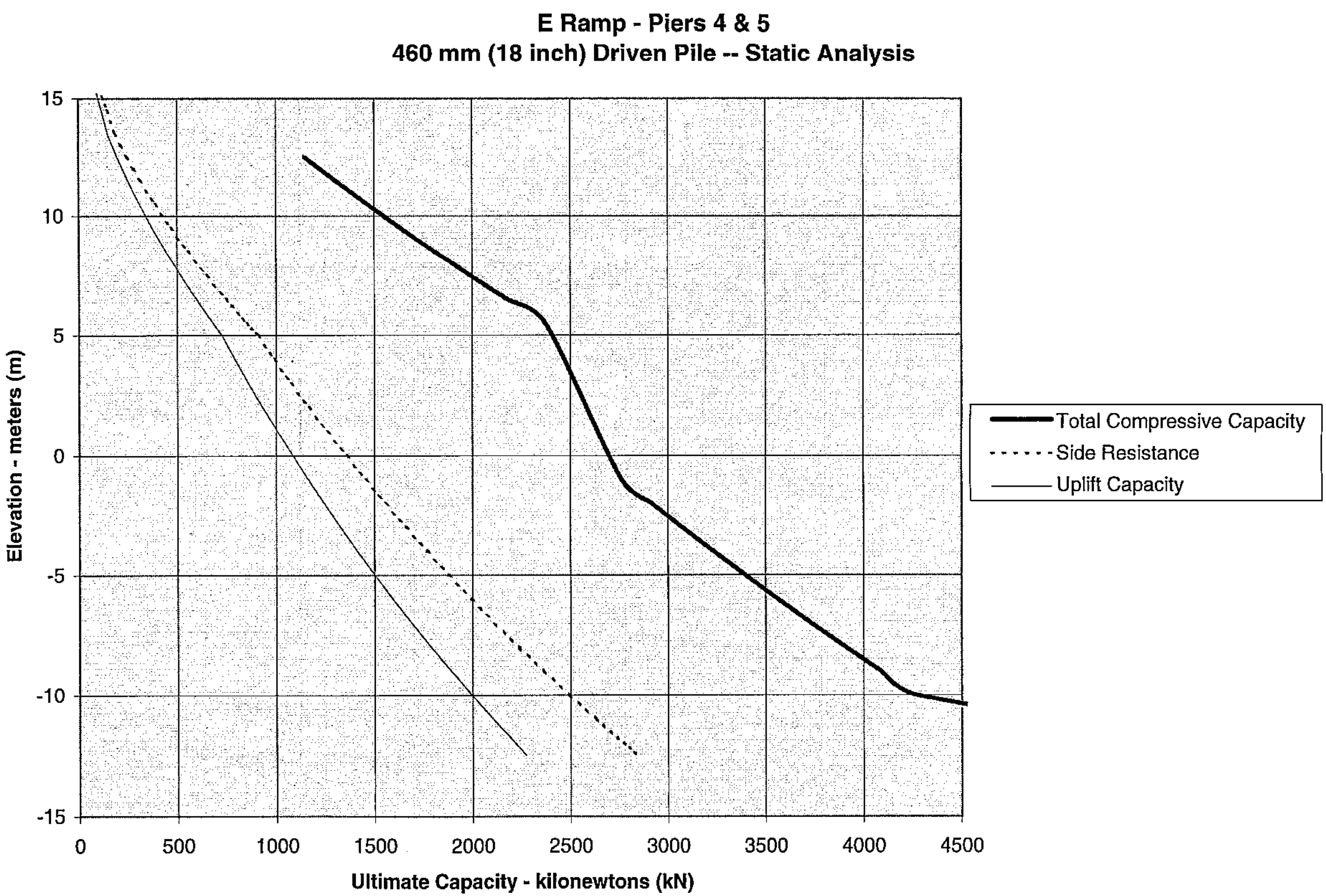


Figure 5-5. Ultimate Pile Capacity for 460 mm (18 inch) Driven, Closed-End Steel Piles for Piers 4 & 5 at E Ramp – Static Analysis

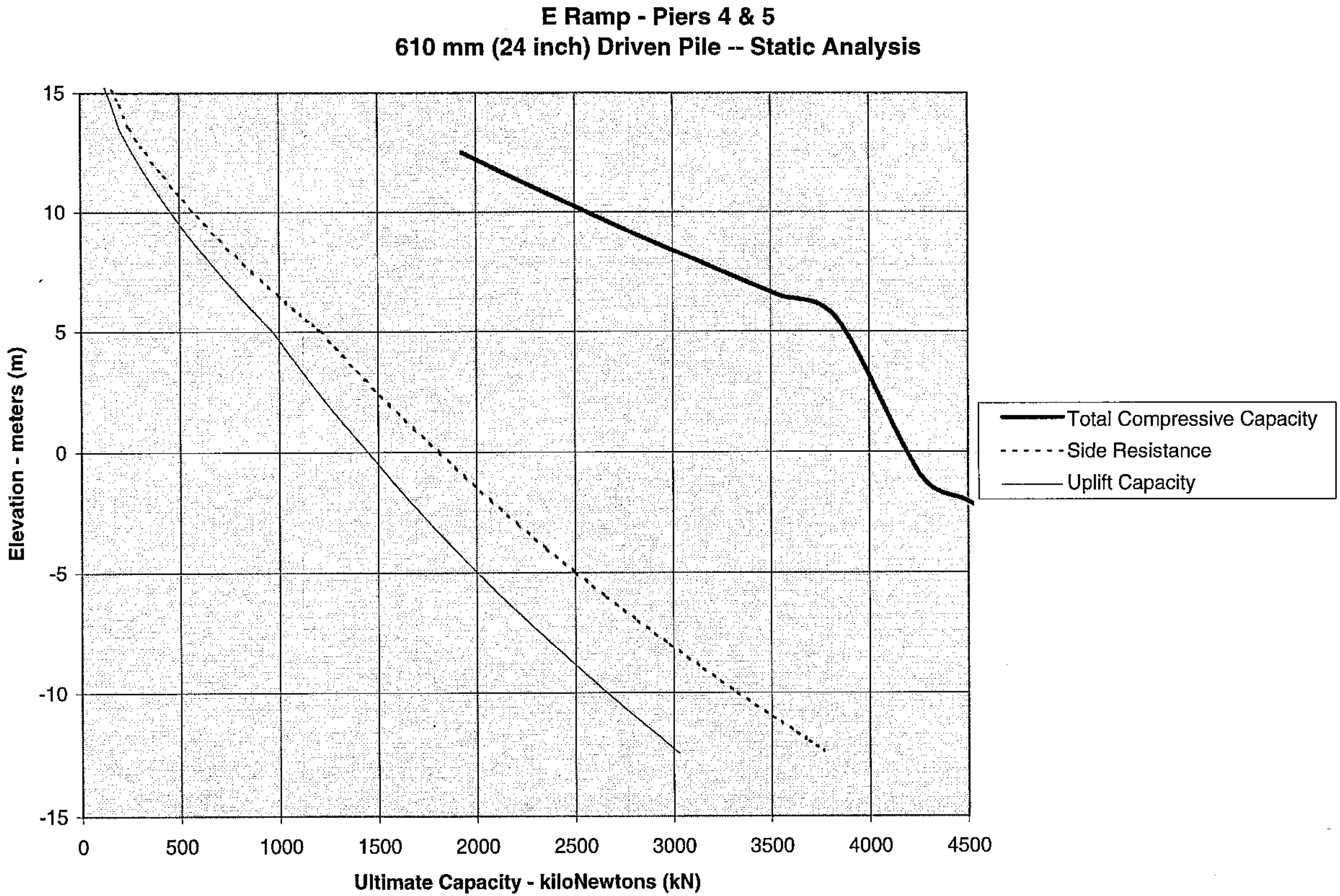


Figure 5-6. Ultimate Pile Capacity for 610 mm (24 inch) Driven, Closed-End Steel Piles for Piers 4 & 5 at E Ramp – Static Analysis

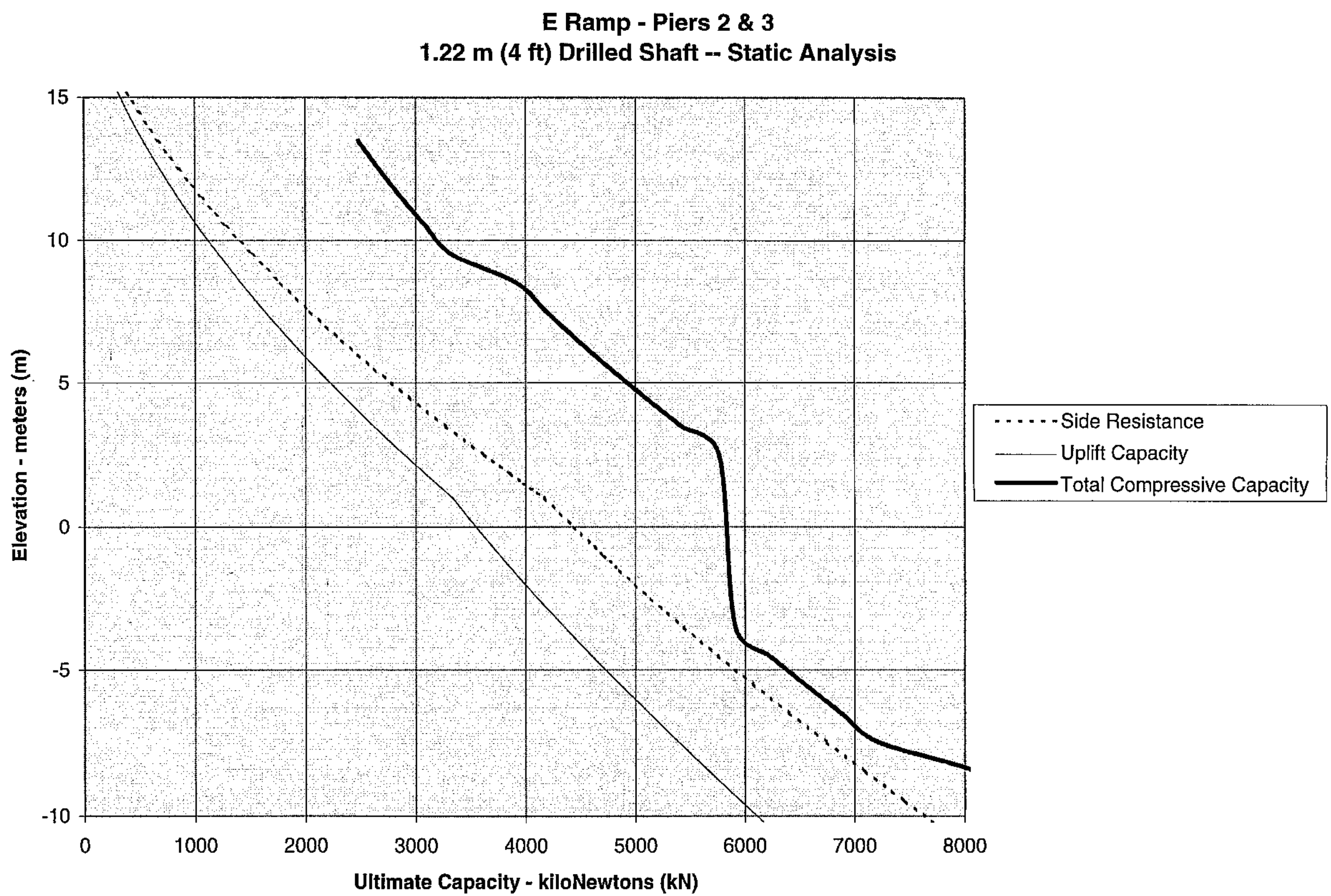


Figure 5-7. Ultimate Capacity of 1.22 m (4 ft) Drilled Shaft for Piers 2 & 3 at E Ramp – Static Analysis

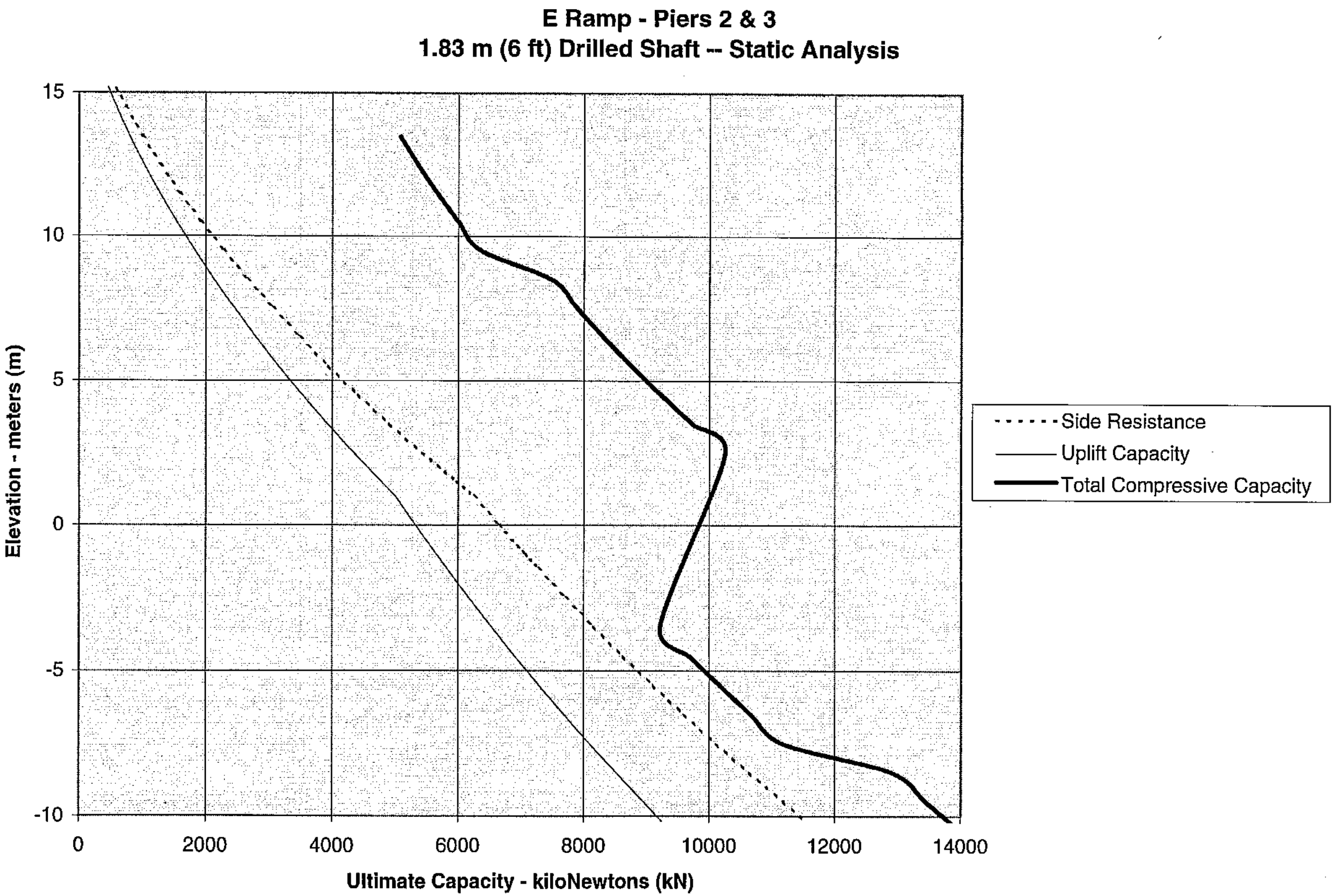


Figure 5-8. Ultimate Capacity of 1.83 m (6 ft) Drilled Shaft for Piers 2 & 3 at E Ramp – Static Analysis

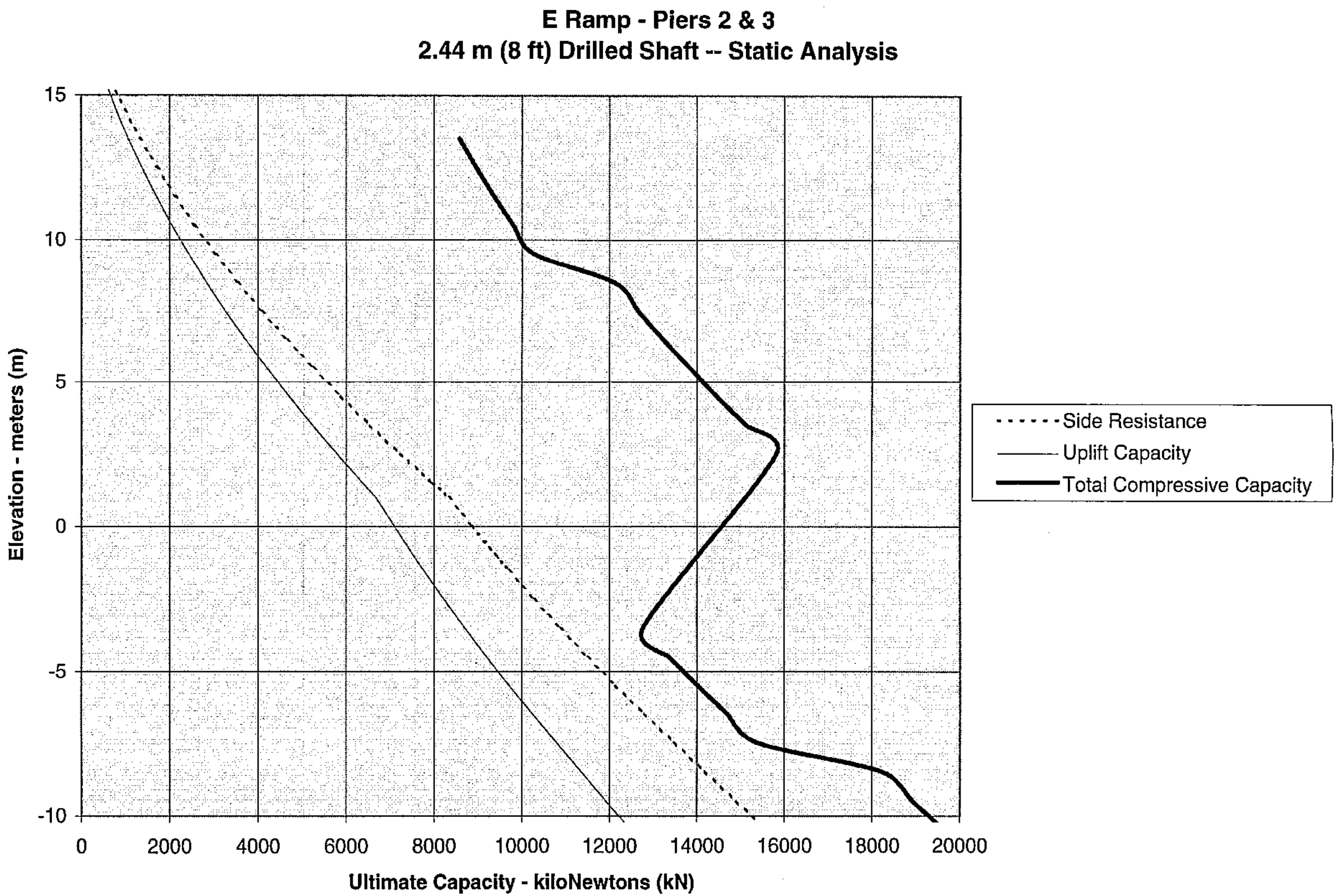


Figure 5-9. Ultimate Capacity of 2.44 m (8 ft) Drilled Shaft for Piers 2 & 3 at E Ramp – Static Analysis

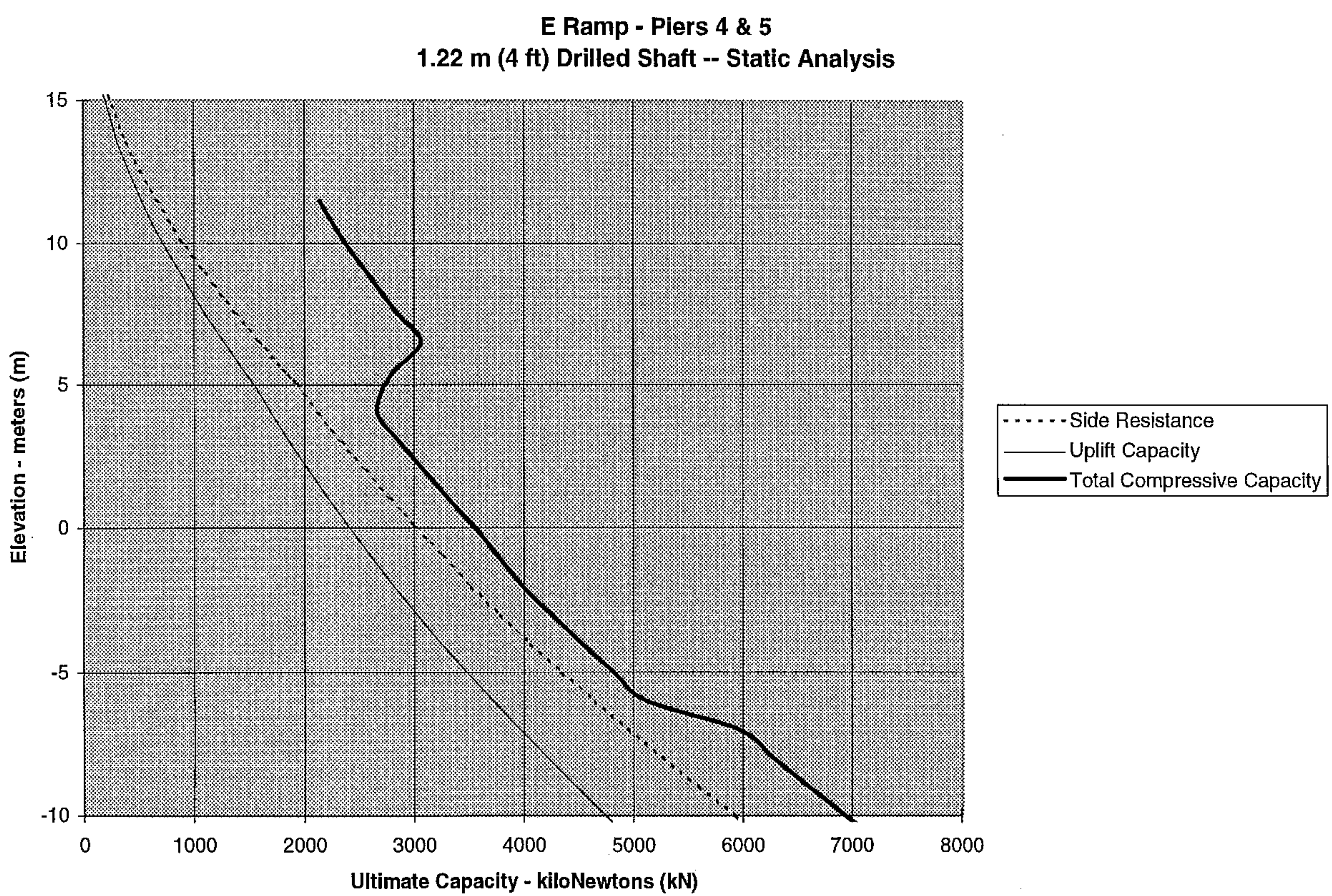


Figure 5-10. Ultimate Capacity of 1.22 m (4 ft) Drilled Shaft for Piers 4 & 5 at E Ramp – Static Analysis

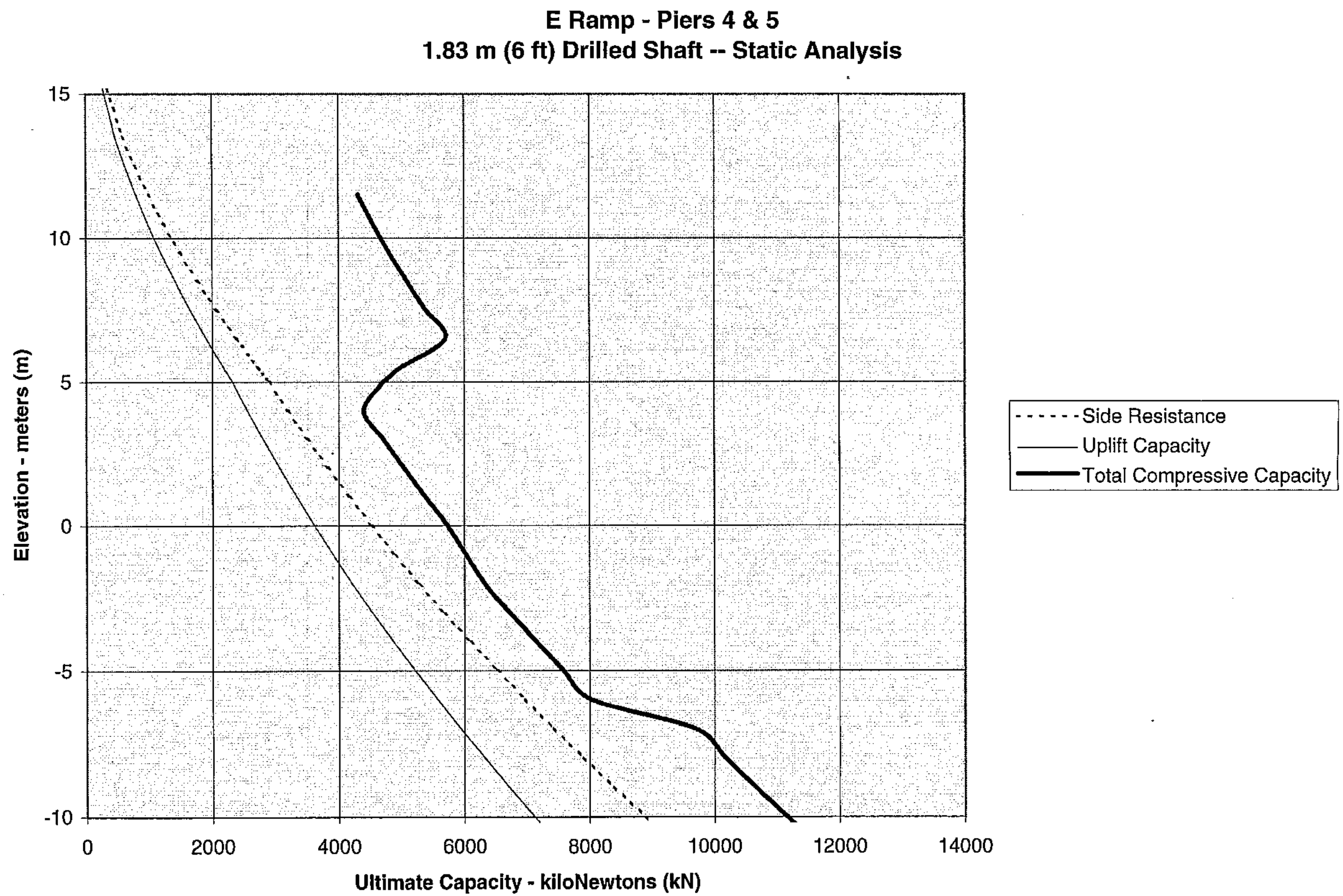


Figure 5-11. Ultimate Capacity of 1.83 m (6 ft) Drilled Shaft for Piers 4 & 5 at E Ramp – Static Analysis

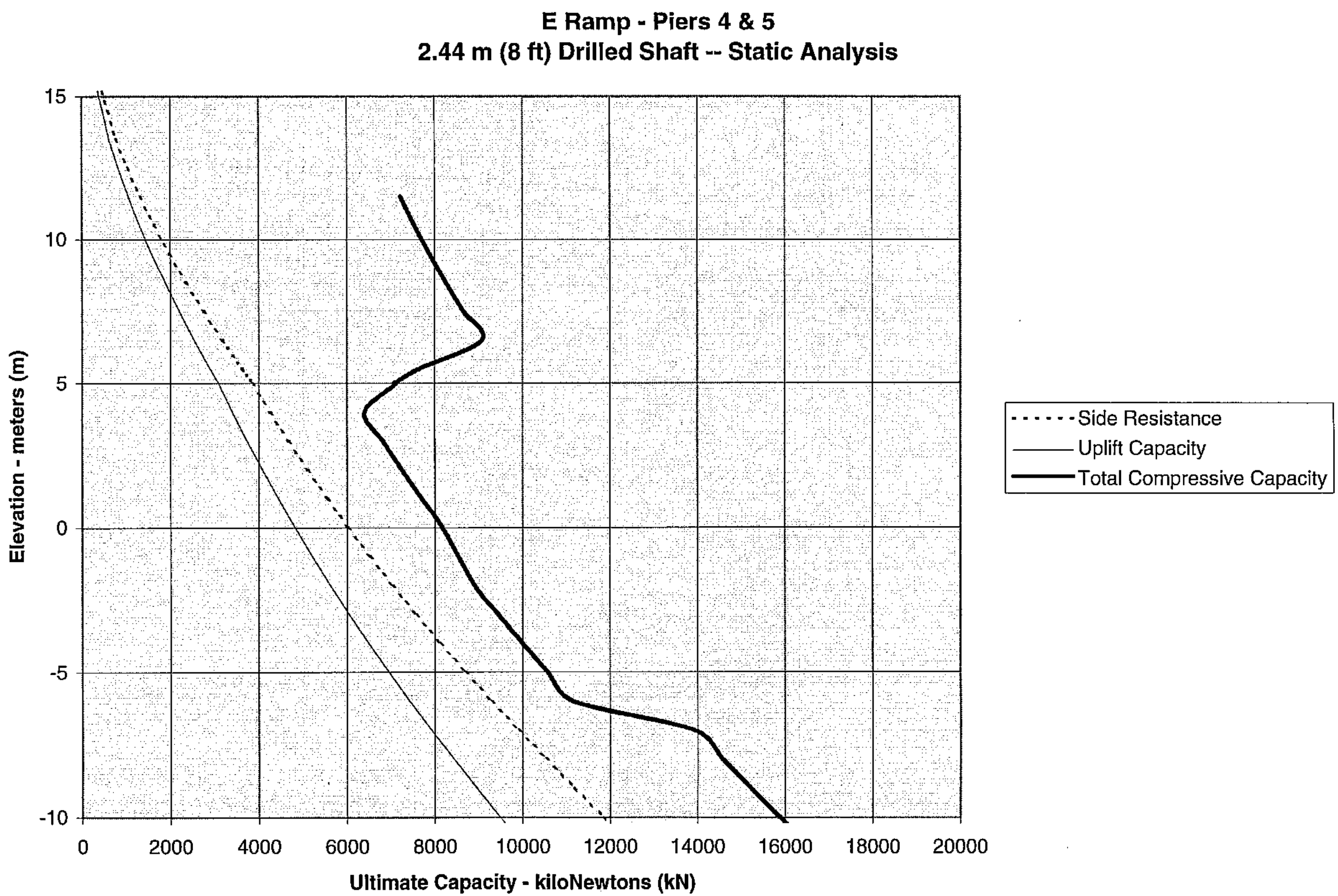


Figure 5-12. Ultimate Capacity of 2.44 m (8 ft) Drilled Shaft for Piers 4 & 5 at E Ramp – Static Analysis

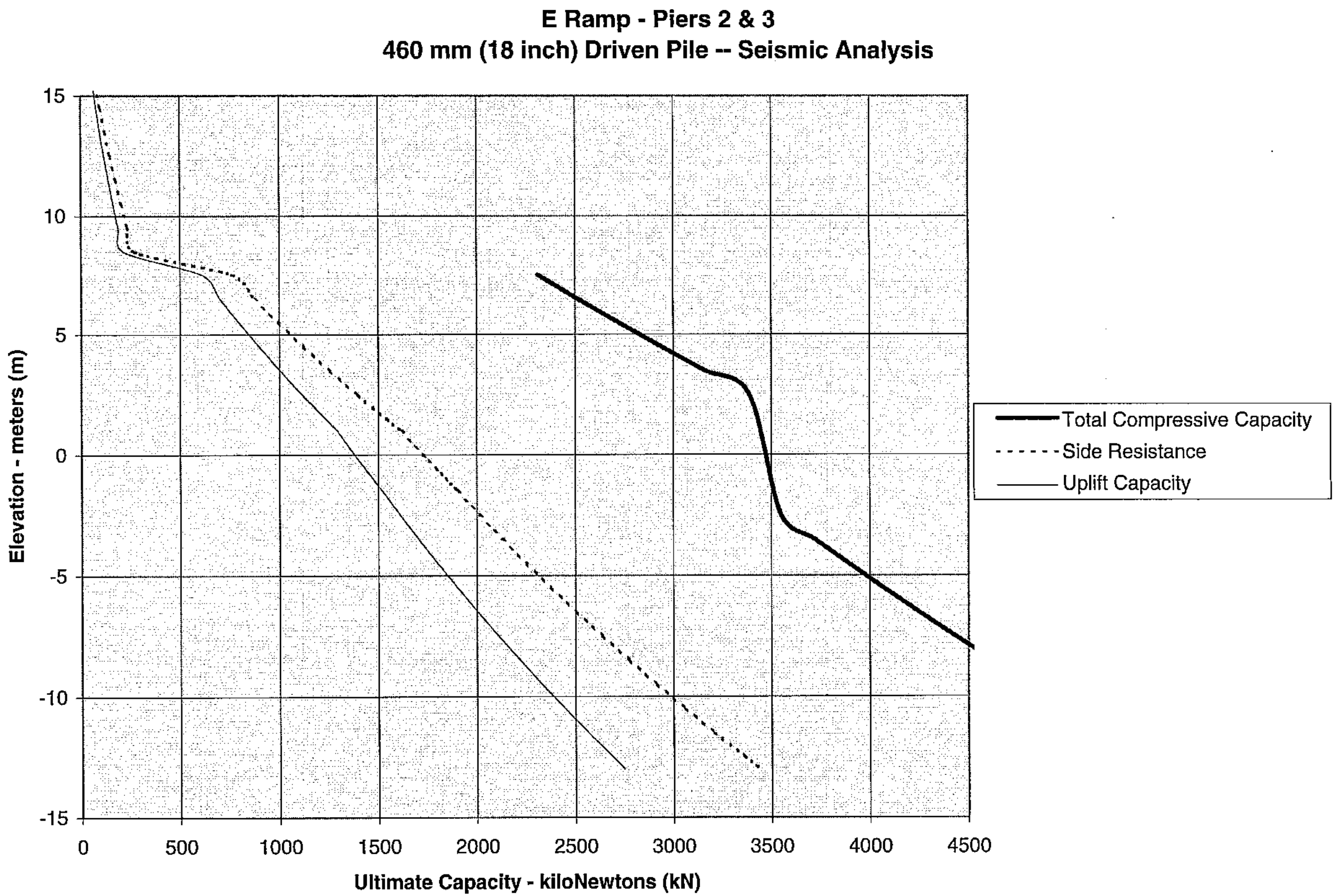


Figure 5-13. Ultimate Pile Capacity for 460 mm (18 inch) Driven, Closed-End Steel Piles for Piers 2 & 3 at E Ramp – Seismic Analysis

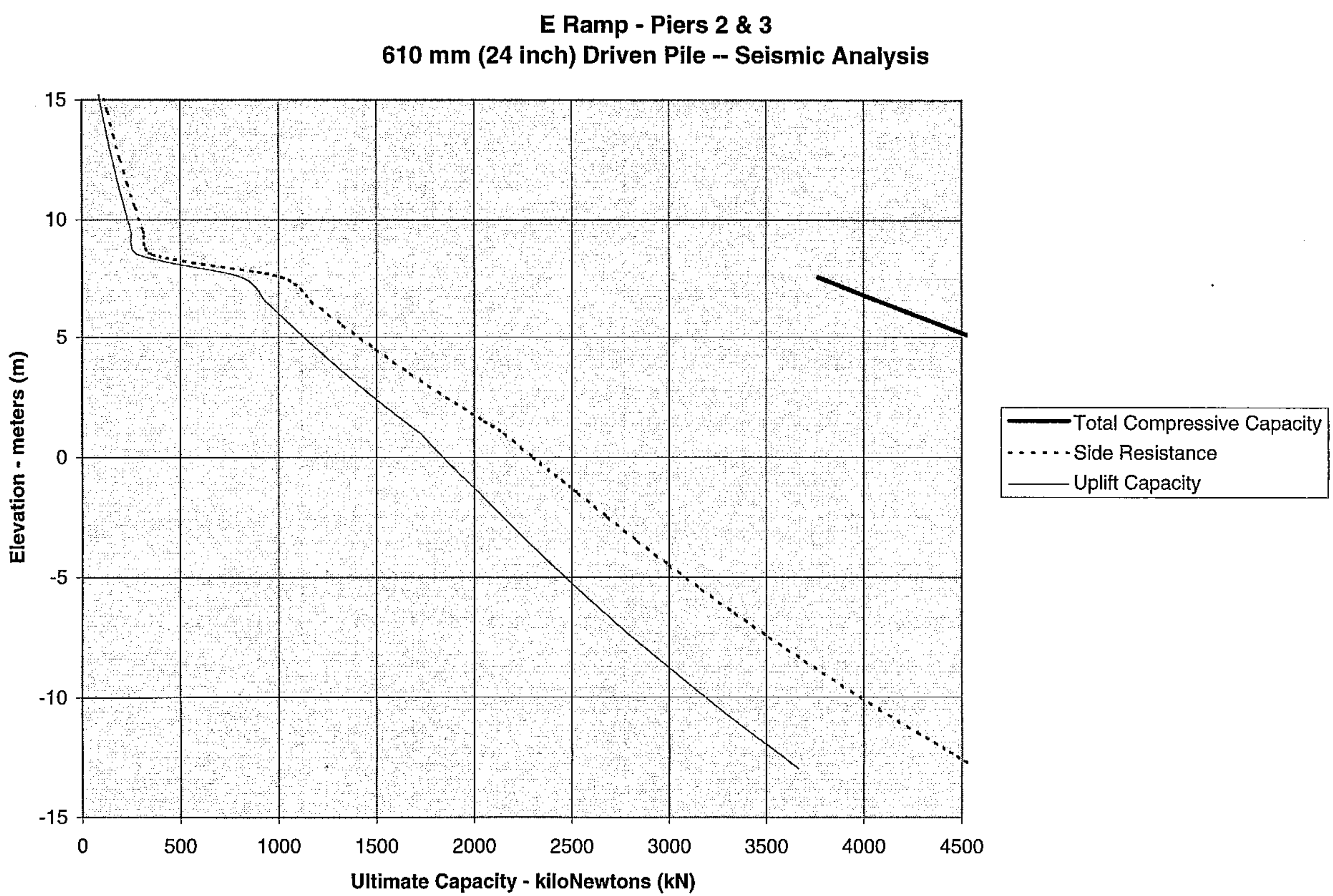


Figure 5-14. Ultimate Pile Capacity for 610 mm (24 inch) Driven, Closed-End Steel Piles for Piers 2 & 3 at E Ramp – Seismic Analysis

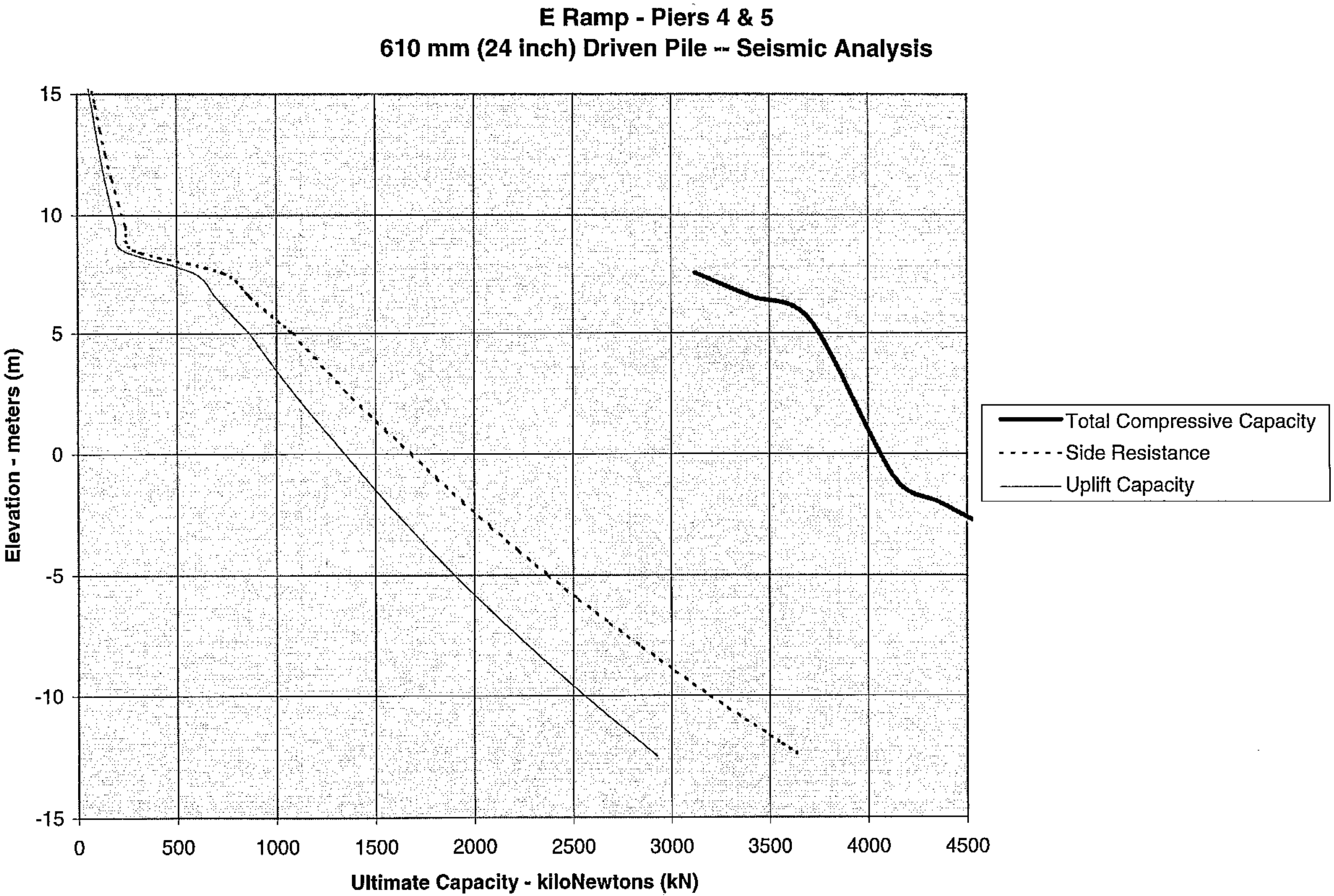


Figure 5-15. Ultimate Pile Capacity for 460 mm (18 inch) Driven, Closed-End Steel Piles for Piers 4 & 5 at E Ramp – Seismic Analysis

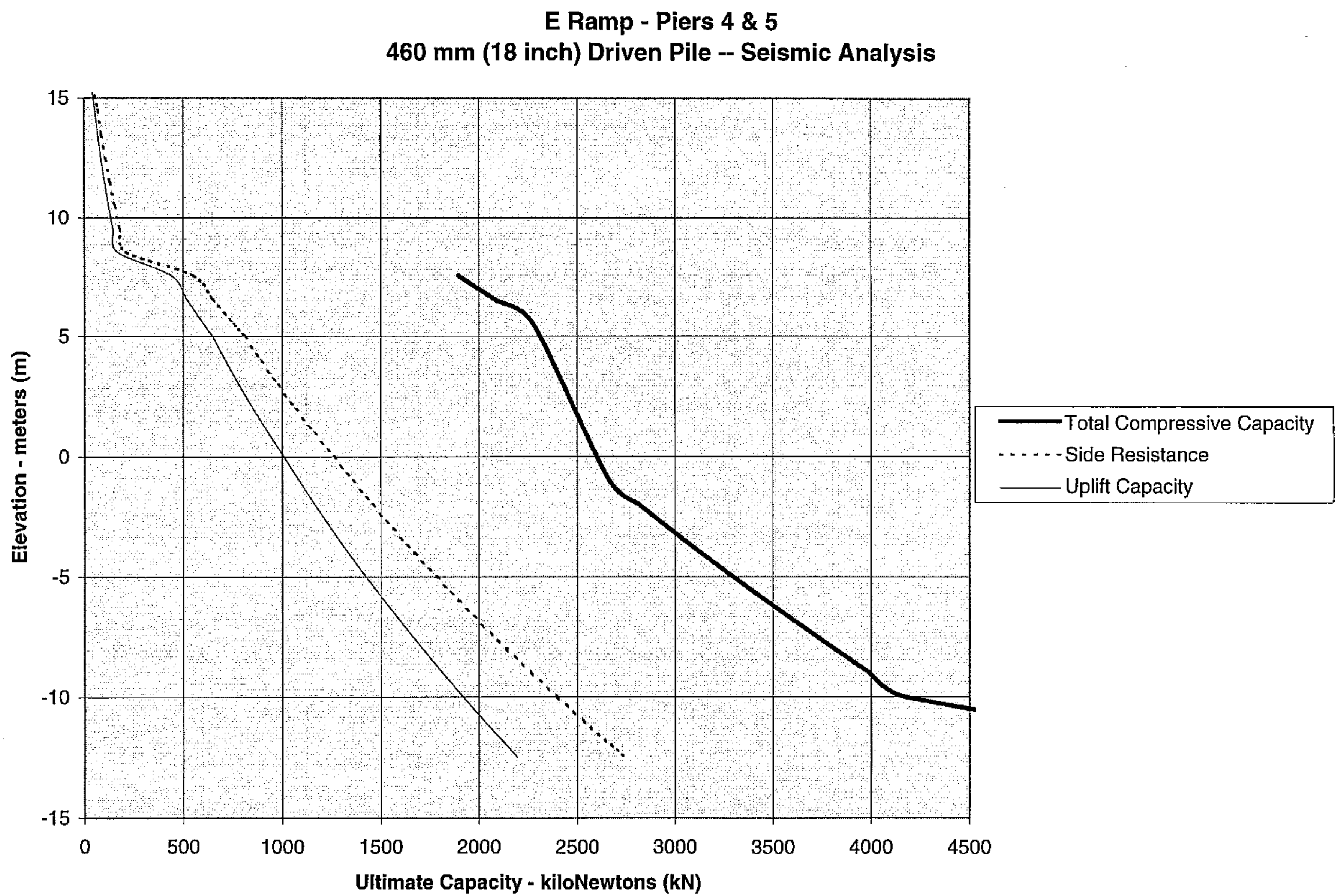


Figure 5-16. Ultimate Pile Capacity for 610 mm (24 inch) Driven, Closed-End Steel Piles for Piers 4 & 5 at E Ramp – Seismic Analysis

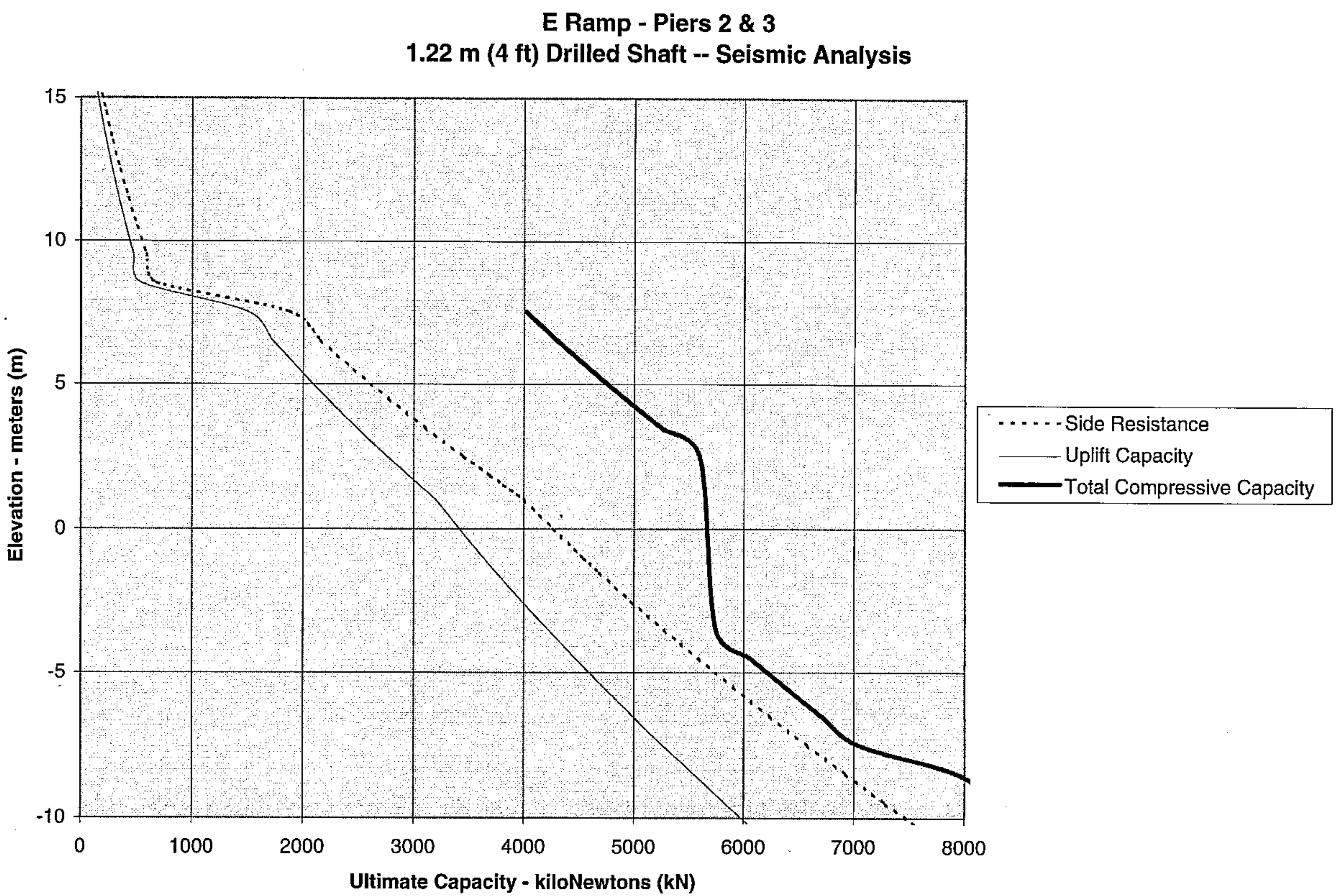


Figure 5-17. Ultimate Capacity of 1.22 m (4 ft) Drilled Shaft for Piers 2 & 3 at E Ramp – Seismic Analysis

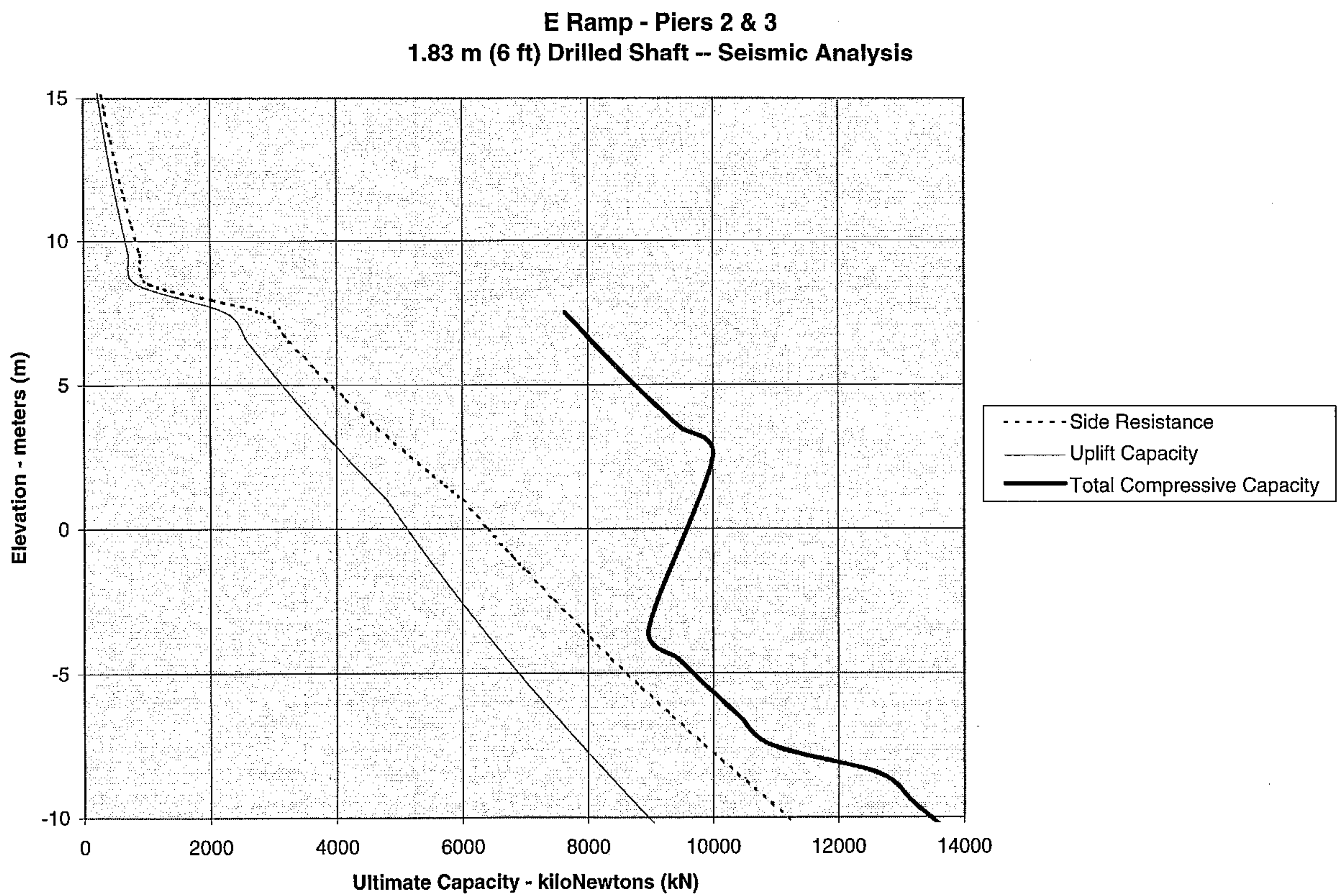


Figure 5-18. Ultimate Drilled Shaft Capacity of 1.83 m (6 ft) Drilled Shaft for Piers 2 & 3 at E Ramp – Seismic Analysis

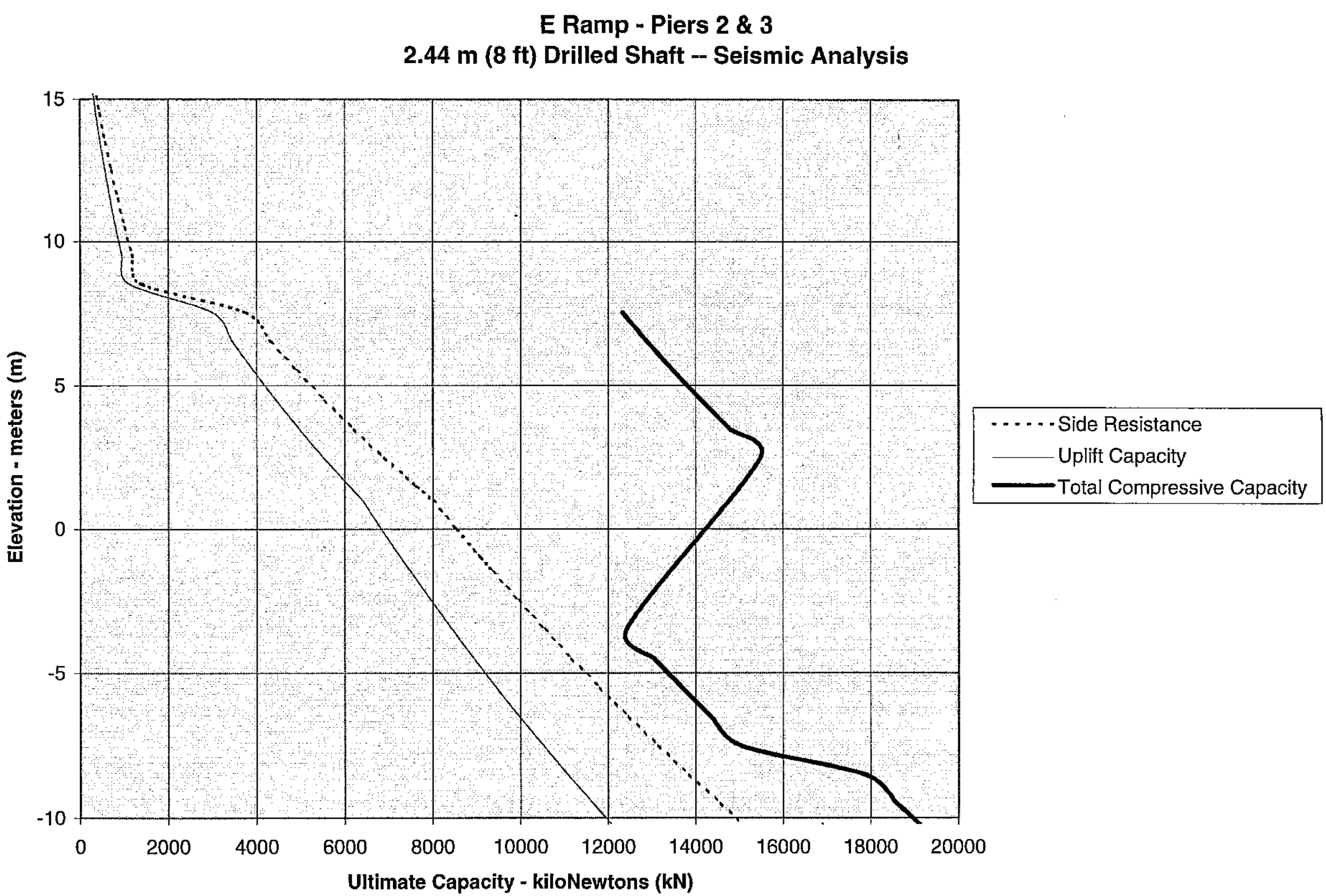


Figure 5-19. Ultimate Drilled Shaft Capacity of 2.44 m (8 ft) Drilled Shaft for Piers 2 & 3 at E Ramp – Seismic Analysis

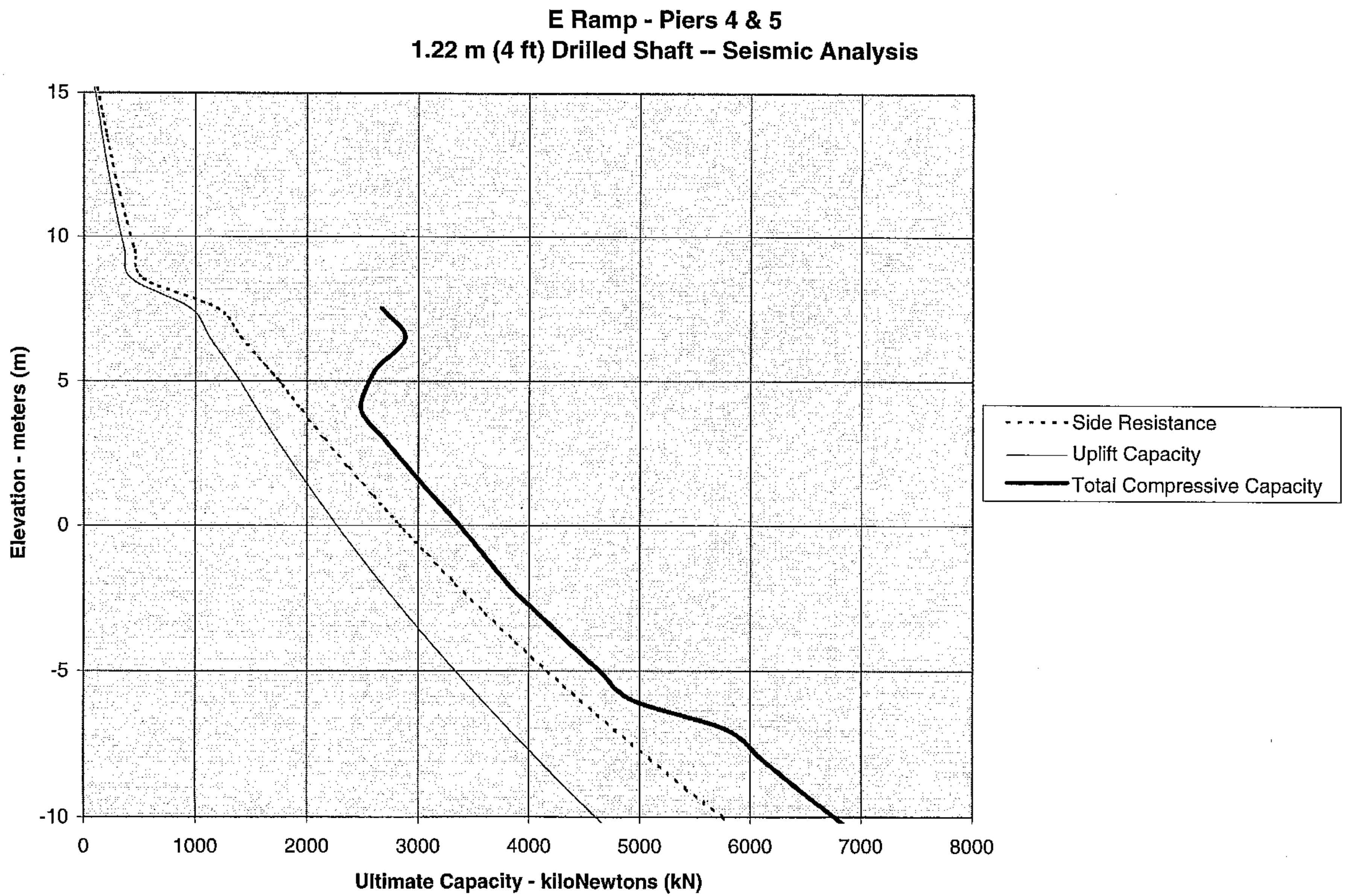


Figure 5-20. Ultimate Capacity of 1.22 m (4 ft) Drilled Shaft for Piers 4 & 5 at E Ramp – Seismic Analysis

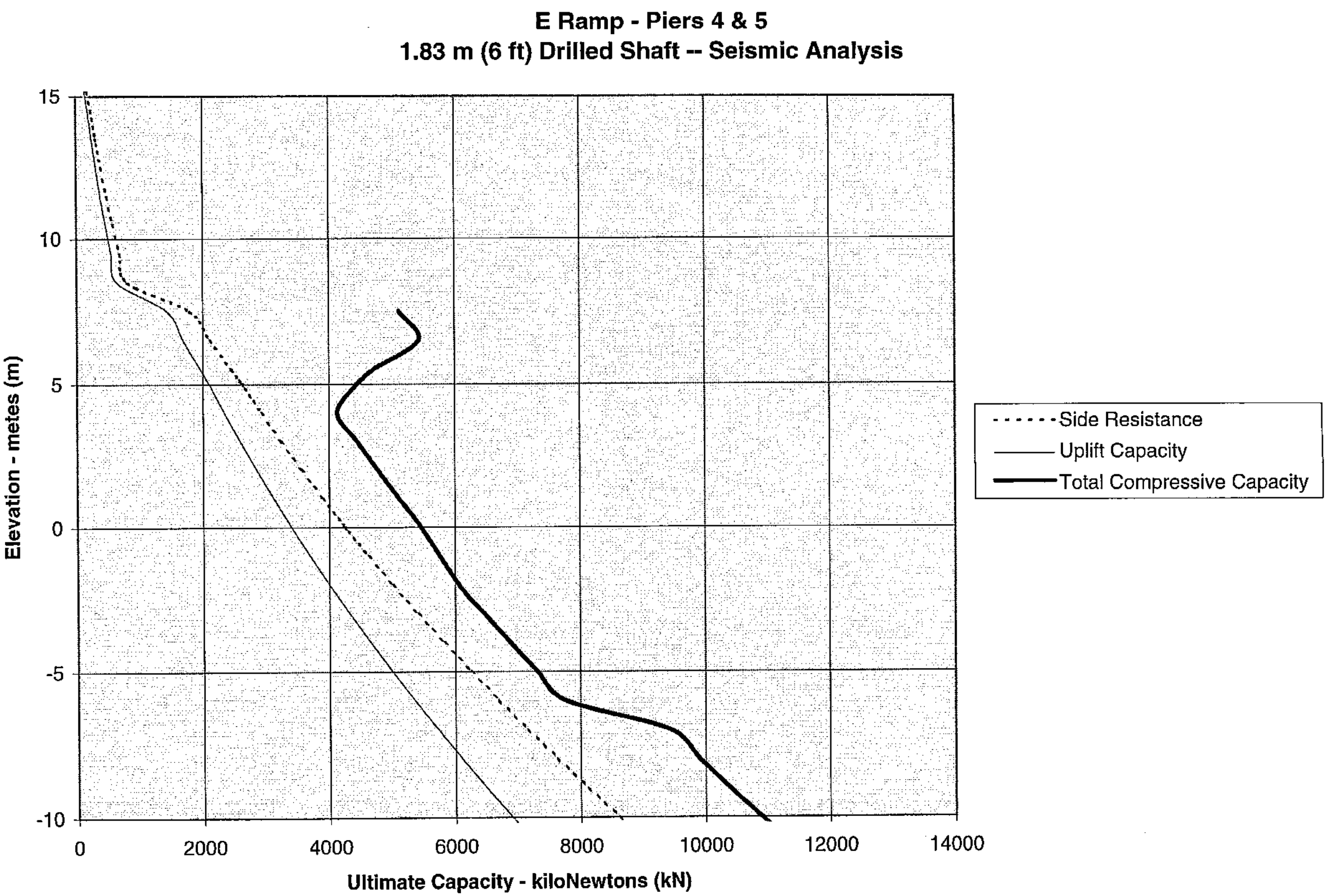


Figure 5-21. Ultimate Drilled Shaft Capacity of 1.83 m (6 ft) Drilled Shaft for Piers 4 & 5 at E Ramp – Seismic Analysis

1997 SOIL TEST HOLE LOGS

FOR

E RAMP

CH2M HILL

Proj. No.: 116184.G4

SOIL TEST HOLE LOG

Project: SR-167, CS 1765/6, OL-2305 Drilling Contractor: WSDOT
Drilling Method & Equipment: Longyear BK-80 Truck Mounted Rig Logger: R. Devulapally/Terra

Depth Below Surface (FT)	Sample: Interval	Sample: Number and Type	Sample: Recovery (FT)	Standard Penetration Test Results 6"-6"-6" (N)	Soil Description: Soil name, uscs group symbol, color, moisture content, relative density or consistency, soil structure, mineralogy	Comments: Depth of casing, drilling rate, drilling fluid loss, tests and instrumentation
Elevation: 23m					Location: Sta. 158+72m; Offset 9.5m R of CL	**Test Hole H-4-97**
Start: 12/08/97					Finish: 12/11/1997	Water Level: 20.7m
						Sheet 1 of 4
						Started drilling at 12:30 pm using wash rotary with 4" casing
5.0	4.0 - 5.5	S-1	0.5	10-9-11	SAND, (SP), black to gray, medium dense, with trace of clay and gravel (FILL)	
10.0	9.0 - 10.5	S-2	0.6	7-10-10	SAND, (SP), gray, medium dense, with some gravel and silt	
						Driller reports drilling through gravels and cobbles. Losing considerable return water
15.0	14.0 - 15.5	S-3	0.2	9-8-7	GRAVEL, (GP), black, medium dense, some sand	
20.0	19.0 - 20.5	S-4	0.9	2-4-3	SILT, (ML), black, wet, medium dense, some sand	
25.0	24.0 - 25.5	S-5	1.1	10-13-16	SILTY SAND, (SM), fine, black, wet, medium dense	
						Stopped drilling at 4:00 pm Resumed drilling at 7:30 am on 12/09/97
30.0	29.0 - 30.5	S-6	1.0	5-5-14	SILTY SAND, (SM/ML), fine, black, medium dense, some silt seams, with organics	
35.0	34.0 -	S-7	1.5	9-9-11	SAND, (SP), fine, black, wet, medium	

NOTES:

1) Test hole located below N-E Ramp bridge 14' west and 9' north of southern pier on shoulder of ramp from EB 18 to NB 167
2) All blowcounts recorded with WSDOT automatic hammer
3) Water encountered in test hole at approximate elevation 20.7m on 12/09/97 and 20.6m on 12/10/97

1 foot = 0.3048 meters
1 inch = 25.4 millimeters

Proj. No.: 116184.G4

SOIL TEST HOLE LOG

Project: SR-167, CS 1765/6, OL-2305 Drilling Contractor: WSDOT
Drilling Method & Equipment: Longyear BK-80 Truck Mounted Rig Logger: R. Devulapally/Terra

Depth Below Surface (FT)	Sample: Interval	Sample: Number and Type	Sample: Recovery (FT)	Standard Penetration Test Results 6"-6"-6" (N)	Soil Description: Soil name, uscs group symbol, color, moisture content, relative density or consistency, soil structure, mineralogy	Comments: Depth of casing, drilling rate, drilling fluid loss, tests and instrumentation
Elevation: 23m					Location: Sta. 158+72m; Offset 9.5m R of CL	**Test Hole H-4-97**
Start: 12/08/97					Finish: 12/11/97	Water Level: 20.7m
						Sheet 2 of 4
	35.5				dense, some silt seams, trace of gravel	Driller reports encountering gravels at about 38'
40.0	39.0 - 40.5	S-8	0.7	15-25-18	SAND AND GRAVEL, (SP/GP), black, dense, some silt	
45.0	44.0 - 45.5	S-9	0.5	12-10-10	SAND AND GRAVEL, (SP/GP), black, medium dense, some silt	
50.0	49.0 - 50.5	S-10	1.3	17-19-11	SAND, (SP), fine to medium, black, dense, with gravel and some silt	
55.0	54.0 - 55.5	S-11	1.2	5-11-16	SAND, (SP), fine, black, medium dense, some gravel	
						Switch to 3" casing with wire line wash rotary
60.0	59.0 - 60.5	S-12	1.1	31-31-41	SAND, (SP), black, very dense, with gravel and trace of silt	
65.0	64.0 - 65.5	S-13	1.0	14-12-11	SAND, (SP), fine to medium, black, medium dense, some gravel, trace of silt	Trace of clay
70.0	69.0 -	S-14	1.0	22-16-16	SAND, (SP), fine to medium, black, dense,	

NOTES:

1 foot = 0.3048 meters
1 inch = 25.4 millimeters

CH2M HILL

Proj. No.: 116184.G4

SOIL TEST HOLE LOG

Project: SR-167, CS 1765/6, OL-2305 Drilling Contractor: WSDOT

Drilling Method & Equipment: Longyear BK-80 Truck Mounted Rig Logger: R. Devulapally/Terra

Elevation: 23m | Location: Sta. 158+72m; Offset 9.5m R of CL | **Test Hole H-4-97**

Start: 12/08/97 | Finish: 12/11/97 | Water Level: 20.7m

Sheet 3 of 4

Depth Below Surface (FT)	Sample Interval	Sample Number and Type	Sample Recovery (FT)	Standard Penetration Test Results 6"-6"-6" (N)	Soil Description: Soil name, uscs group symbol, color, moisture content, relative density or consistency, soil structure, mineralogy	Comments: Depth of casing, drilling rate, drilling fluid loss, tests and instrumentation
	70.5				some gravel	Stopped drilling at 3:45 pm Resumed drilling at 8:00 am on 12/10/97
75.0	74.0 - 75.5	S-15	1.5	3-5-10	SILTY CLAY, (CL), black, medium stiff (top 12") and SILTY SAND, (SM), fine, black, medium dense, with silt (bottom 6")	Driller notes gravel seam at 77'
80.0	79.0 - 80.5	S-16	1.5	2-2-9	SILTY CLAYEY SAND, (SP/SM/SC), black to gray, medium dense, some gravel	Considerable loss of wash water
85.0	84.0 - 85.5	S-17	1.5	2-2-2	CLAYEY SAND TO SANDY CLAY, (SC), black to gray, loose, with some gravel	
90.0	89.0 - 90.5	S-18	1.5	3-2-3	CLAYEY SAND, (SC), black, loose, with silt and some gravel	
95.0	94.0 - 95.5	S-19	0.4	3-2-3	SAND TO SILTY SAND (SP/SM), black, loose, trace of gravel	
100.0	99.0 - 100.5	S-20	1.5	1-3-2	SAND TO SILTY SAND, (SP/SM), gray to black, loose, some clay and gravel	Loss of return water
105.0	104.0 -	S-21	1.5	7-5-4	SANDY CLAY, (SC), gray, soft, some	Pocket pen = 0.75 tsf

NOTES:

1 foot = 0.3048 meters
1 inch = 25.4 millimeters

CH2M HILL

Proj. No.: 116184.G4

SOIL TEST HOLE LOG

Project: SR-167, CS 1765/6, OL-2305 Drilling Contractor: WSDOT

Drilling Method & Equipment: Longyear BK-80 Truck Mounted Rig Logger: R. Devulapally/Terra

Depth Below Surface (FT)	Sample: Interval	Sample: Number and Type	Sample: Recovery (FT)	Standard Penetration Test Results 6"-6"-6" (N)	Soil Description: Soil name, uscs group symbol, color, moisture content, relative density or consistency, soil structure, mineralogy	Comments: Depth of casing, drilling rate, drilling fluid loss, tests and instrumentation
Elevation: 23m					Location: Sta. 158+72m; Offset 9.5m R of CL	**Test Hole H-4-97**
Start: 12/08/97					Finish: 12/11/97	Water Level: 20.7m
						Sheet 4 of 4
	105.5.				gravel and silt	
110.0	109.0 - 110.5	S-22	1.3	15-8-11	CLAYEY SAND TO SANDY CLAY, (SC), gray, medium dense, medium stiff, some gravel	
115.0	114.0 - 115.5	S-23	1.5	4-3-11	SILTY CLAYEY SAND, (SC/ML), gray, medium dense, some gravel	
120.0	119.0 - 120.5	S-24		10-23-35	SAND AND GRAVEL, (SP/GP), gray to black, very dense, some silt	
					END SOIL TEST HOLE AT 120.5 FEET	Stopped drilling at 4:30 pm on 12/10/97
125.0						
130.0						
135.0						
140.0						

NOTES:

Installed piezometer. Total length 50'. Bottom 2' solid casing (1"). 10' of screen with 1/32" slot at 1/4" spacing. Sand pack located from 36' to 50' bgs. Top 38' of piezometer 1" solid pvc casing. Hole plug from 10' to 36' bgs. Quickcrete from 0 to 10'. Locking cap located at ground surface.

1 foot = 0.3048 meters

1 inch = 25.4 millimeters

Proj. No.: 116184.G4

SOIL TEST HOLE LOG

Project: SR-167, CS 1765/6, OL-2305 Drilling Contractor: WSDOT

Drilling Method & Equipment: Longyear BK-80 Truck Mounted Rig Logger: R. Devulapally/Terra

Depth Below Surface (FT)	Sample: Interval	Sample: Number and Type	Sample: Recovery (FT)	Standard Penetration Test Results 6"-6"-6" (N)	Soil Description: Soil name, uscs group symbol, color, moisture content, relative density or consistency, soil structure, mineralogy	Comments: Depth of casing, drilling rate, drilling fluid loss, tests and instrumentation
Elevation: 21m					Location: Sta. 159+25m; Offset 11.6m R of CL	**Test Hole H-5-97**
Start: 12/04/97					Finish: 12/04/97	Water Level: 18.5m
						Sheet 1 of 2
						Start drilling at 9:00 am on 12/04/97 using 4" HAS
5.0	3.0 - 4.5	S-1	0.8	6-3-2	SAND AND GRAVEL, (SP/GP), brown, moist, loose, trace of silt (FILL)	
10.0	8.0 - 9.5	S-2	0.6	3-8-9	SAND, (SP), black, moist, medium dense some gravel and silt	
15.0	13.0 - 14.5	S-3	0.4	1-0-3	SILTY SAND, (SM), fine, black, moist, loose, wood fragments	
20.0	18.0 - 19.5	S-4	1.3	3-4-5	SILTY SAND, (SM), fine, black, wet, loose, wood fragments	Water encountered at about 18'
25.0	23.0 - 24.5	S-5	1.5	2-1-3	SILTY SAND TO SAND, (SP/SM), black, wet, very loose, silt seam from 23.5' to 24'	
30.0	28.0 - 29.5	S-6	1.5	5-3-2	SILTY SAND, (SM), black, wet, loose, some silt seams	
35.0	33.0 - 34.5	S-7	1.5	4-4-2	SAND, (SP), fine, black, moist, loose, trace of silt, wood fragments	

NOTES:

1) Test hole located below N-E Ramp bridge 10' west of 2nd pier from north and 4' south of southern pier c north abutment and 4' south of barrier between WB 18 & ramp off WB18
2) All blowcounts recorded with WSDOT automatic hammer
3) Water encountered in test hole at approximate elevation 15.5m. By end of drilling water at 18.5m.

1 foot = 0.3048 meters
1 inch = 25.4 millimeters

CH2M HILL

Proj. No.: 116184.G4
SOIL TEST HOLE LOG

Project: SR-167, CS 1765/6, OL-2305 Drilling Contractor: WSDOT
Drilling Method & Equipment: Longyear BK-80 Truck Mounted Rig Logger: R. Devulapally/Terra

Depth Below Surface (FT)	Sample: Interval	Sample: Number and Type	Sample: Recovery (FT)	Standard Penetration Test Results 6"-6"-6" (N)	Soil Description: Soil name, uscs group symbol, color, moisture content, relative density or consistency, soil structure, mineralogy	Comments: Depth of casing, drilling rate, drilling fluid loss, tests and instrumentation
Elevation: 21m					Location: Sta. 159+25m; Offset 11.6m R of CL	**Test Hole H-5-97**
Start: 12/04/97					Finish: 12/04/97	Water Level: 18.5m
						Sheet 2 of 2
40.0	38.0 - 40.0	S-8	1.0	1-2-3-5	SILTY SAND, (SM), fine to medium, black, wet, loose, wood fragments	
45.0	43.0 - 45.0	S-9	1.5	4-6-6-3	SILTY SAND, (SM), fine to medium, black, wet, medium dense, wood fragments, some peat	
50.0	48.0 - 50.0	S-10	1.5	5-6-8-8	SILTY SAND, (SM), fine to coarse, black, medium dense, wood fragments, some gravel	
					END OF SOIL TEST HOLE AT 50.0 FEET	Stopped drilling at 12:15 pm on K2112/4/1997
55.0						
60.0						
65.0						
70.0						

NOTES:

1 foot = 0.3048 meters
1 inch = 25.4 millimeters

Proj. No.: 116184.G4

SOIL TEST HOLE LOG

Project: SR-167, CS 1765/6, OL-2305 Drilling Contractor: WSDOT

Drilling Method & Equipment: Longyear BK-80 Truck Mounted Rig Logger: R. Devalpally/Terra

Depth Below Surface (FT)	Sample Interval	Sample Number and Type	Sample Recovery (FT)	Standard Penetration Test Results 6"-6"-6" (N)	Soil Description: Soil name, uscs group symbol, color, moisture content, relative density or consistency, soil structure, mineralogy	Comments: Depth of casing, drilling rate, drilling fluid loss, tests and instrumentation
Elevation: 23m					Location: Sta. 159+40m; Offset 8.5m of CL	**Test Hole H-6-97**
						Water Level: 21m
Start: 12/15/97					Finish: 12/16/97	Sheet 1 of 3
						Start drilling at 2:00 pm using 4" HAS
5.0	5.0 - 6.5	S-1	1.0	5-10-15	SILTY SAND TO SAND (SM/SP), fine to medium, brown to gray, moist to dry, trace of gravel and vegetation (fill)	Driller notes cobbles blow 5'
10.0	10.0 - 10.5	S-2	1.1	5-16-20	SAND AND GRAVEL, (SP/GW), fine to medium sand, fine to coarse gravel, brown to gray, moist to dry, dense, some silt, trace of vegetation (FILL)	Wet cuttings at 13' but no groundwater
15.0	15.0 - 16.5	S-3	0.4	6-2-3	SANDY SILT, (ML), fine, dark gray, loose, occasional seams of fine sand, trace of vegetation	Groundwater at 18'
20.0	20.0 - 21.5	S-4	1.5	6-7-9	SAND, (SP), fine, black, wet, medium dense, some silt, 1" gravel particle	
25.0	25.0 - 26.5	S-5	0.8	3-4-4	SAND, (SP), fine, black, wet, loose, some silt	Heave in augers. Washed out
30.0	30.0 - 31.5	S-6	0.5	4-4-4	SAND, (SP), fine to medium, black, loose, with wood fragments	
35.0	35.0 -	S-7	0.8	8-12-10	SAND, (SP), fine to medium, black,	Stopped drilling at 4:00 pm

NOTES:

1) Test hole located below N-E Ramp bridge 17' west of west column at toe of abutment slope
2) All blowcounts recorded with WSDOT automatic hammer
3) Water encountered in test hole at approximate elevation 17.5m on 12/15/97. Measured at 21.5m on 12/16/97

1 foot = 0.3048 meters
1 inch = 25.4 millimeters

Proj. No.: 116184.G4

SOIL TEST HOLE LOG

CH2M HILL

Project: SR-167, CS 1765/6, OL-2305 Drilling Contractor: WSDOT

Drilling Method & Equipment: Longyear BK-80 Truck Mounted Rig Logger: R. Devalpally/Terra

Depth Below Surface (FT)	Sample: Interval	Sample: Number and Type	Sample: Recovery (FT)	Standard Penetration Test Results 6"-6"-6" (N)	Soil Description: Soil name, uscs group symbol, color, moisture content, relative density or consistency, soil structure, mineralogy	Comments: Depth of casing, drilling rate, drilling fluid loss, tests and instrumentation
Elevation: 23m					Location: Sta. 159+40m; Offset 8.5m of CL	Test Hole H-6-97
Start: 12/15/97					Finish: 12/16/97	Water Level: 21m
						Sheet 2 of 3
	36.5				medium dense, with 4" wood fragment	Resumed drilling at 8:00 am
40.0	40.0 - 41.5	S-8	0.6	8-7-9	SAND, (SP), fine to coarse, black to brown, medium dense, some gravel, traces of wood	Heave in augers. Washed out to sample
45.0	45.0 - 46.5	S-9	0.4	3-6-10	SAND, (SP), fine to medium, black, medium dense, with wood fragments	Heave in augers. Washed out to sample
50.0	50.0 - 51.5	S-10	1.1	1-2-4	SAND, (SP), fine to medium, black, loose, trace of gravel, fine sandy silt seam from 51.2' to 51.5', with wood fragments	Heave in augers. Washed out to sample. Seated spoon 4" to sample
55.0	55.0 - 56.5	S-11	0.7	1-4-10	SAND, (SP), fine to medium, black, medium dense, some gravel	Heave in augers. Washed out to sample
60.0	60.0 - 61.5	S-12	0.8	2-3-8	SAND, (SP), fine, black, medium dense, some silt	Introduced column of water to resist heave
65.0	65.0 - 66.5	S-13	1.4	2-4-4	SAND TO SILTY SAND, (SP/SM), fine, black, loose	Introduced column of water to resist heave
70.0	70.0 -	S-14	0.4	2-14-18	SAND, (SP), fine, black, dense, some silt	Introduced column of

NOTES:

1 foot = 0.3048 meters
1 inch = 25.4 millimeters

Proj. No.: 116184.G4

SOIL TEST HOLE LOG

Project: SR-167, CS 1765/6, OL-2305 Drilling Contractor: WSDOT

Drilling Method & Equipment: Longyear BK-80 Truck Mounted Rig Logger: R. Devalpally/Terra

Depth Below Surface (FT)	Sample: Interval	Sample: Number and Type	Sample: Recovery (FT)	Standard Penetration Test Results 6"-6"-6" (N)	Soil Description: Soil name, uscs group symbol, color, moisture content, relative density or consistency, soil structure, mineralogy	Comments: Depth of casing, drilling rate, drilling fluid loss, tests and instrumentation
Elevation: 23m					Location: Sta. 159+40m; Offset 8.5m of CL	**Test Hole H-6-97**
Start: 12/15/97					Finish: 12/16/97	Water Level: 21m
						Sheet 3 of 3
	71.5					water to resist heave
75.0	75.0 - 76.5	S-15	1.5	4-6-4	SANDY SILT, (ML), fine, black, loose, seam of fine to medium sand	Introduced column of water to resist heave
80.0	80.0 - 81.5	s-16	0.6	7-8-12	SANDY SILTY, (ML), fine, black, medium dense	Heave in auger. Washing out. Seated 12" after repeated attempts to wash out heave
					END OF SOIL TEST HOLE AT 81.5 FEET	Stopped drilling at 1:30 pm on 12/16/97
85.0						
90.0						
95.0						
100.0						
105.0						

NOTES:

1 foot = 0.3048 meters
1 inch = 25.4 millimeters

CH2M HILL

Proj. No.: 116184.G4

SOIL TEST HOLE LOG

Project: SR-167, CS 1765/6, OL-2305 Drilling Contractor: WSDOT

Drilling Method & Equipment: Longyear BK-80 Truck Mounted Rig Logger: R. Devulapally/Terra

Depth Below Surface (FT)	Sample: Interval	Sample: Number and Type	Sample: Recovery (FT)	Standard Penetration Test Results 6"-6"-6" (N)	Soil Description: Soil name, uscs group symbol, color, moisture content, relative density or consistency, soil structure, mineralogy	Comments: Depth of casing, drilling rate, drilling fluid loss, tests and instrumentation
Elevation: 30m					Location: Sta. 159+57m; Offset 8.8m R of CL	**Test Hole H-7-97**
Start: 12/04/97					Finish: 12/05/97	Water Level: 24m
						Sheet 1 of 2
						Started drilling at 2:30 pm on 12/04/97 using 4" HAS
5.0	4.0 - 5.5	S-1	0.4	8-13-11	SAND AND GRAVEL, (SP/GP), black, medium dense, with vegetation (FILL)	
10.0	9.0 - 10.5	S-2	1.2	5-16-14	SAND AND GRAVEL, (SP/GP), fine, brown, medium dense (FILL)	
15.0	14.0 - 15.5	S-3	1.5	3-7-8	SAND AND GRAVEL, (SP/GP), brown to gray, medium dense, trace of clay, some silt (FILL)	
20.0	19.0 - 20.5	S-4	0.2	2-4-5	SAND AND GRAVEL, (SP/GP), brown, loose (FILL)	Water measured at 20' on 12/05/97
25.0	24.0 - 25.5	S-5	1.5	7-16-30	CLAYEY SAND, (SC), brown to gray, moist, dense to hard, with gravel	
30.0	29.0 - 30.5	S-6	1.3	14-13-11	SILT, (ML), gray, medium dense, compact, some clay, trace of sand and gravel	Resumed drilling at 8:00 am
35.0	34.0 -	S-7	1.2	30-27-39	SAND, (SP), brown to black, very dense,	

NOTES:

1) Test hole located about 9' east of the west edge of the north abutment of NB 167 over SR18 north abutment and 4' south of barrier between WB 18 & ramp off WB18
2) All blowcounts recorded withWSDOT automatic hammer
3) Water encountered in test hole at approximate elevation 24m at 8:00 am on 12/05/97

1 foot = 0.3048 meters
1 inch = 25.4 millimeters

Proj. No.: 116184.G4
SOIL TEST HOLE LOG

Project: SR-167, CS 1765/6, OL-2305   Drilling Contractor: WSDOT

Drilling Method & Equipment: Longyear BK-80 Truck Mounted Rig   Logger: R. Devulapally/Terra

Depth Below Surface (FT)	Sample: Interval	Sample: Number and Type	Sample: Recovery (FT)	Standard Penetration Test Results 6"-6"-6" (N)	Soil Description: Soil name, uscs group symbol, color, moisture content, relative density or consistency, soil structure, mineralogy	Comments: Depth of casing, drilling rate, drilling fluid loss, tests and instrumentation
Elevation: 30m					Location: Sta. 159+57m; Offset 8.8m R of CL	**Test Hole H-7-97**
Start: 12/04/97					Finish: 12/05/97	Water Level: 24m
						Sheet 2 of 2
	35.5				some clay and gravel	
40.0	39.0 - 40.5	S-8	0.2	5-8-8	SAND, (SP), black, wet, medium dense, with gravel	
					END OF SOIL TEST HOLE AT 40.5 FEET	Stopped drilling at 10:00 am on 12/05/97
45.0						
50.0						
55.0						
60.0						
65.0						
70.0						

NOTES:

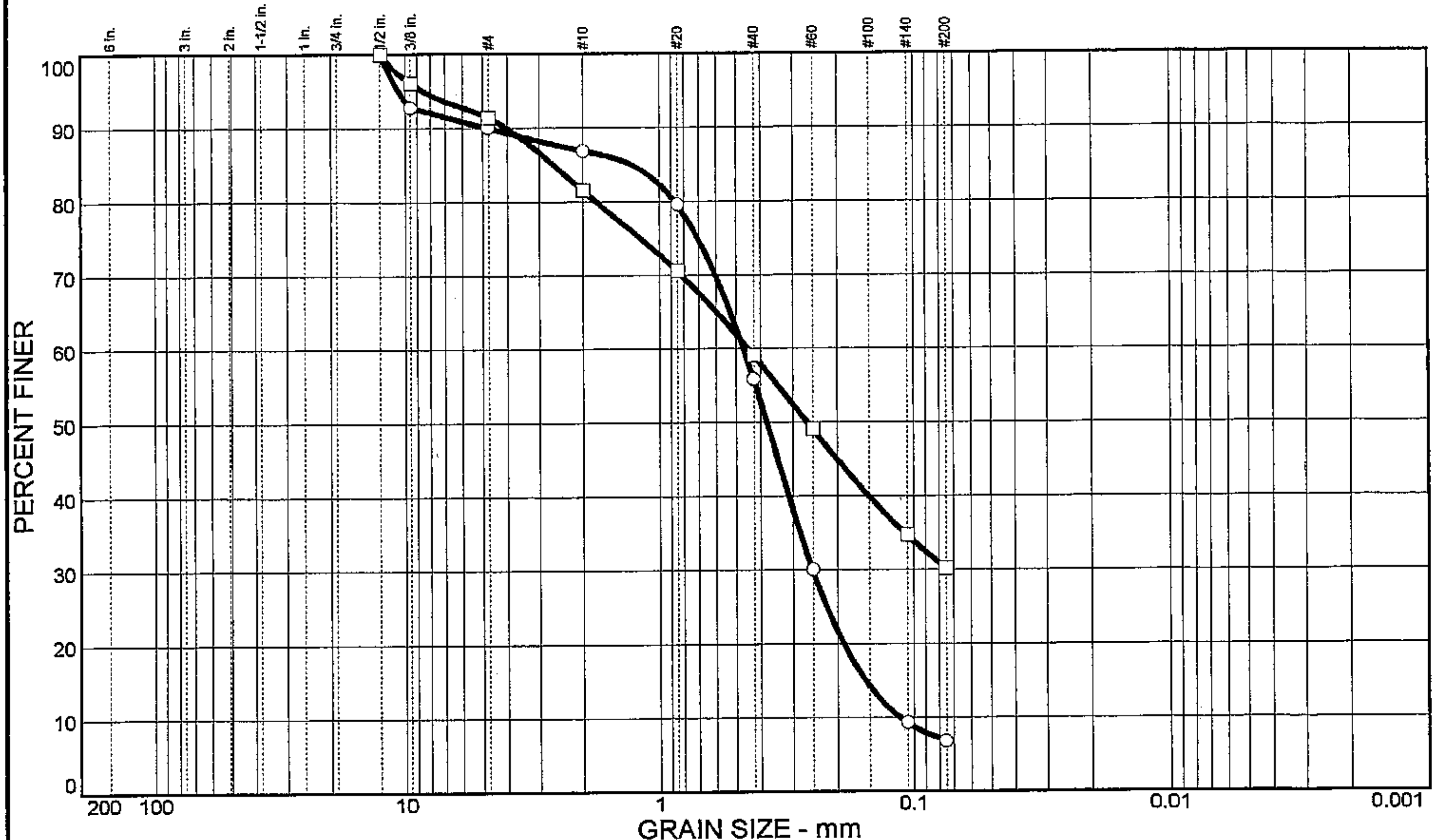
1 foot = 0.3048 meters
1 inch = 25.4 millimeters

1997 LABORATORY TEST DATA

FOR

E RAMP

PARTICLE SIZE DISTRIBUTION TEST REPORT



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○		10.0	83.2			SP-SM			
□		8.6	61.2			SM			

SIEVE inches size	PERCENT FINER ○	□	
.5	100.0	100.0	
.375	92.8	96.1	
	GRAIN SIZE		
D_{60}	0.466	0.446	
D_{30}	0.250		
D_{10}	0.113		
	COEFFICIENTS		
C_c	1.19		
C_u	4.12		

SIEVE number size	PERCENT FINER ○	□	
#4	90.0	91.4	
#10	86.9	81.6	
#20	79.6	70.6	
#40	55.8	59.1	
#60	30.0	49.1	
#140	9.3	34.7	
#200	6.8	30.2	

SOIL DESCRIPTION

○ Poorly graded sand with silt

□ Silty sand

REMARKS:

○

□

○ Source: H-4 Sample No.: 11-S Elev./Depth:

□ Source: H-4 Sample No.: 17-S Elev./Depth:

SOIL TECHNOLOGY, INC.

Client: CH2MHILL

Project: SR-167, OL-2305
E Ramp

Project No.: J-1120

Plate 1

PARTICLE SIZE DISTRIBUTION TEST REPORT

	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○		19.5	74.9			SP-SM			
□		4.0	90.8			SP-SM			

SIEVE inches size	PERCENT FINER ○	□	
1	100.0		
.75	94.6		
.5	92.5	100.0	
.375	86.1	98.6	
	GRAIN SIZE		
D_{60}	0.467	0.457	
D_{30}	0.201	0.269	
D_{10}	0.105	0.101	
	COEFFICIENTS		
C_c	0.82	1.57	
C_u	4.43	4.52	

SIEVE number size	PERCENT FINER ○	□	
#4	80.5	96.0	
#10	75.4	91.9	
#20	66.6	83.6	
#40	57.9	56.0	
#60	38.9	26.6	
#140	10.1	10.6	
#200	5.6	5.2	

SOIL DESCRIPTION

○ Poorly graded sand with silt and gravel

□ Poorly graded sand with silt

REMARKS:

○

□

○ Source: H-5 Sample No.: 9-S Elev./Depth:

□ Source: H-6 Sample No.: 10-S Elev./Depth:

SOIL TECHNOLOGY, INC.

Client: CH2MHILL

Project: SR-167, OL-2305
E Ramp

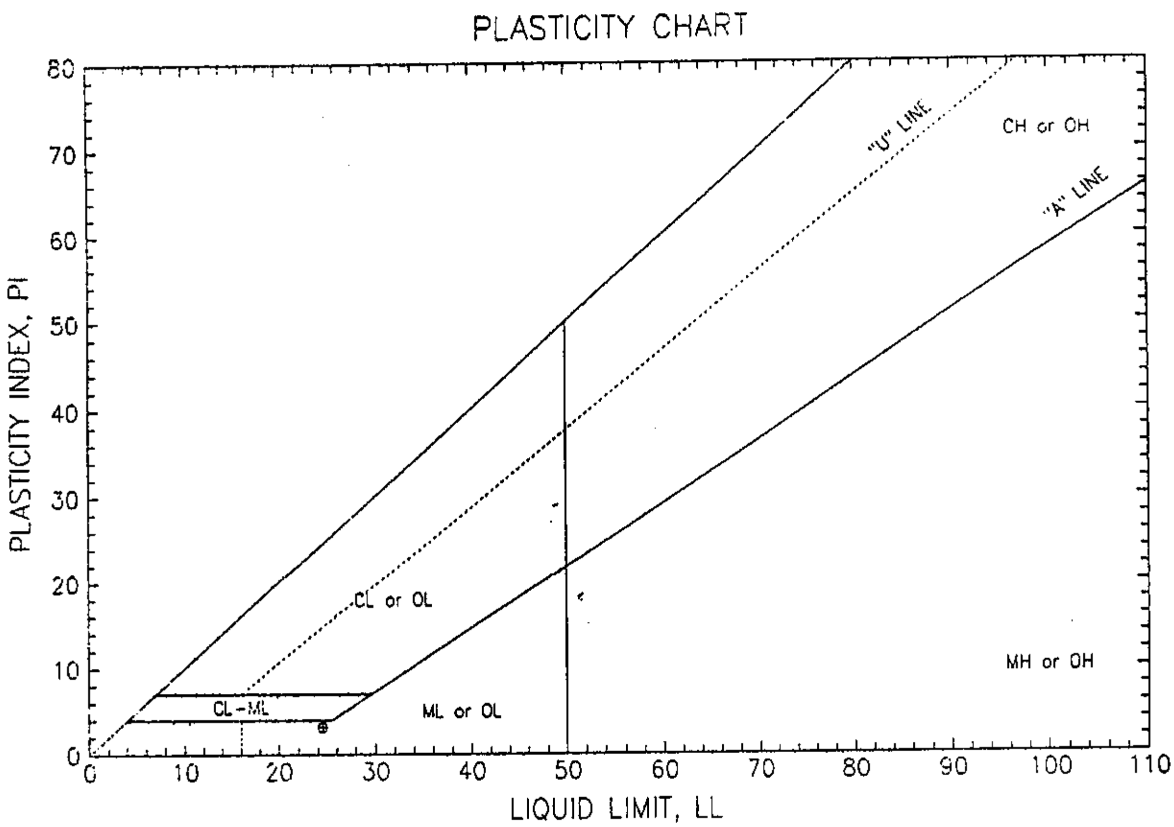
Project No.: J-1120 Plate 2

CH2MHILL
SR-167, OL-2305
E RAMP

Table 1: % Finer than .75 micron

Soil Boring No.	Sample No.	% Finer than .75 micron
H-4	6-S	87
H-5	4-S	24
H-7	5-S	22

Soil Technology, Inc.	Project : E-Ramp Project No. : SR-167,OL-2305 Location : Auburn, WA Date : Mon Jan 12 1998



Symbol	Sample Number	Water Content	Liquid Limit	Plastic Limit	Plasticity Index	Soil Classification
O	H4.21S		24	21	3	(SM) Silty sand

Figure 1

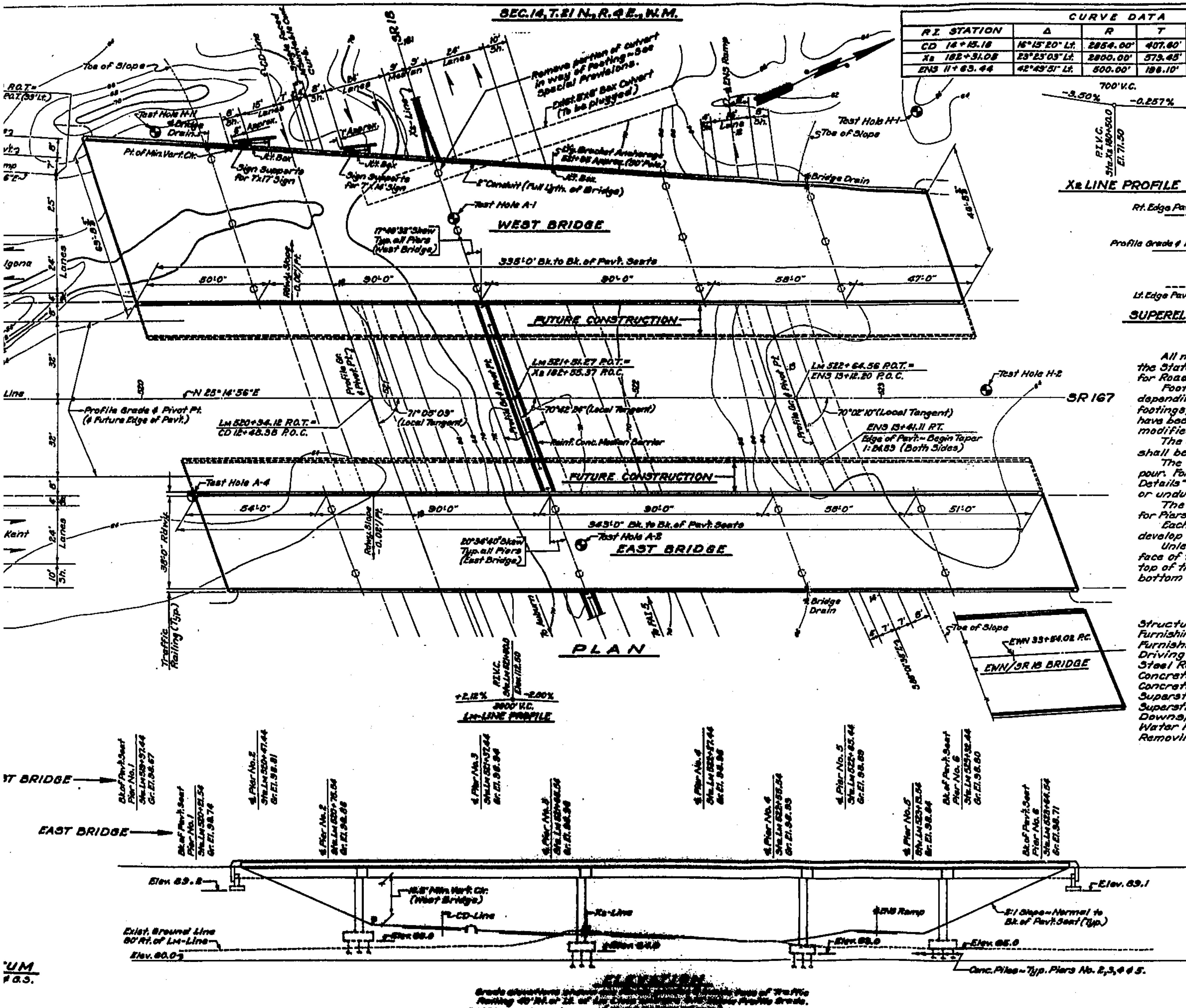
EXISTING DATA

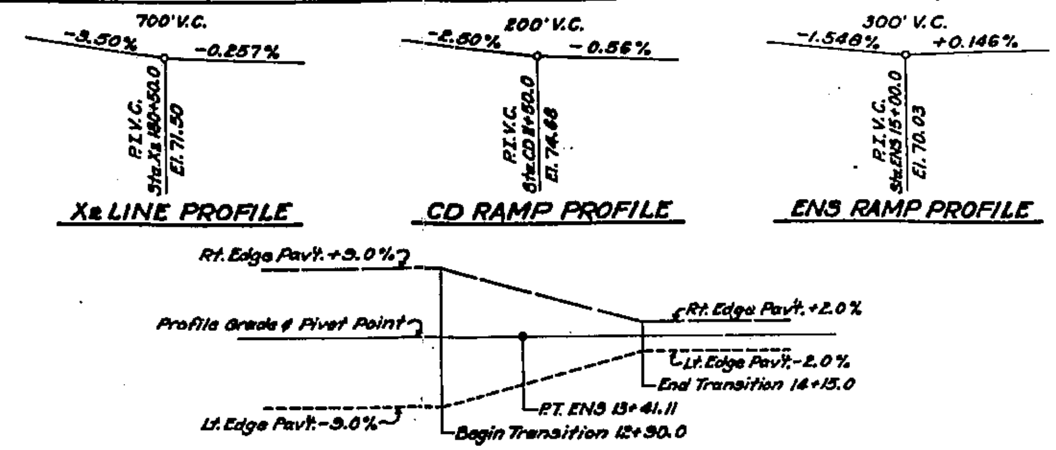
FOR

E RAMP

CURVE DATA						
P.I. STATION	Δ	R	T	L	S (Ft./Ft.)	Bk. Tang.
CD 14+15.18	16°15'20" Lt.	2854.00'	407.60'	809.78'	0.03	S 78°49'08"E
Xa 182+31.08	23°23'03" Lt.	2800.00'	579.45'	1142.78'	0.05	S 71°41'25"E
ENS 11+63.44	42°49'51" Lt.	500.00'	196.10'	373.77	0.03	S 45°11'47"E

STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
WASH	U-021-1(5)	128	170





GENERAL NOTES

All material and work shall be in accordance with the requirements of the State of Washington, Department of Highways, Standard Specifications for Road and Bridge Construction, dated 1969.

Footing elevations and substructure details are subject to change depending upon foundation material encountered. Reinforcing steel for footings, columns and walls shall not be cut until final footing elevations have been determined in the field, and substructure details have been modified as required.

The concrete in the footings of all piers and the walls of Piers No. 1 & 6 shall be Class B mix. All other cast in place concrete shall be Class AX mix.

The roadway slab within each span shall be placed in one continuous pour. For sequence of construction and related information see "Common Details" sheet. Falsework shall be carefully released to prevent impact or undue stresses in the structure.

The maximum design soil pressure per square foot is three (3) tons for Piers No. 1 & 6.

Each pile of Piers No. 2 thru 5 shall be driven to a depth sufficient to develop a minimum load bearing capacity of fifty-five (55) tons.

Unless otherwise shown on the plans, concrete cover measured from the face of the concrete to the face of any reinforcement bar shall be 2" at the top of the roadway slab, 1" at the bottom of the roadway slab, 2½" at the bottom of footings and 1½" at all other locations.

APPROXIMATE QUANTITIES

	EAST BR.	WEST BR.	
Structure Excavation Class A	610	1,100	Cu. Yds.
Furnishing and Driving Concrete Test Piles (55 Ton Capacity)	4	4	Only
Furnishing Concrete Piling (55 Ton Capacity)	2,100	3,050	Lin. Ft.
Driving Concrete Piles (55 Ton Capacity)	60	94	Only
Steel Reinforcing Bars	43,700	80,000	Lbs.
Concrete Class B	160	290	Cu. Yds.
Concrete Class AX	55	75	Cu. Yds.
Superstructure ~ SR 18 Overcrossing, East Bridge	L.S.	—	Lump Sum
Superstructure ~ SR 18 Overcrossing, West Bridge	—	L.S.	Lump Sum
Downspouts	32	50	Lin. Ft.
Water Reducing Additive Est.	560	840	Dollars
Removing Portions of Existing Structure	—	L.S.	Lump Sum

LOADING: HS-20 OR TWO 24^K AXLES @ 4' CTRS.

SR 167 MP 13.77 TO MP 14.73

15TH ST. S.W. TO W. MAIN ST. IN AUBURN

KING COUNTY

SR 18 OVERCROSSINGS

LAYOUT

October 4, 1971

9236

SHEET 128 OF 170 SHEETS

rm 351-003 (H. F. 26.66)
Revised 5-67).

WASHINGTON
STATE HIGHWAY COMMISSION
DEPARTMENT OF HIGHWAYS

Copy to

LOG OF TEST BORING

S.H. S.R. 167 Section West Auburn Interch. SR 167 Job No. L-3598
Hole No. A-4 Sub Section SR 18 O'Xing E. Bridge Pier #1 Cont. Sec. 176503
Station L^M 520+21 Offset 40' Rt. ℄ Ground El. 64
Type of Boring Wash Bore Casing 3" X 115' W.T. El. 64
Inspector Date April 20 to 28, 1971 Sheet 1 of 5

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
			A B C U-1	SILT - Brown, organic
			D E F	PEAT - Dark brown
			G	
				SAND - fine to medium, gray, layered with
			1 Std.	
			2 Pen	SILT, scattered organic matter
5	7		5	
			8 2	Possible scattered wood fragments &/or
				logs
10			A B C U-3	
			D E	
			4	
			5 Std.	
	14		9 Pen	
			10 4	
15			U-5	
			C D E F	
			8	
			7 Std.	
	17		10 Pen	
			5 6	
			U-7	

SR 167 MP 13.77 to MP 14.73
15th St. S.W. to W. Main St. in Auburn
U-021-1(3) - 1971

HWY Form 351-C03-a (H. F. 26.66-A).
Revised 5-67.

Hole No. A-4 Sub Section East Bridge Pier #1 Sheet 2 of 5

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
			C D E U-7	SAND - fine to medium, gray, layered with
			10	silt, scattered organic matter
	30		15 15 Std. Pen	possible scattered wood fragments & or
			9 8	logs
25			B U-9	
			C D	
	20		10 10 Std. 10 Pen 9 10	
30			A B U-11	
	20		9 11 Std. 9 Pen 7 12	
				SANDY GRAVEL & GRAVELLY SAND, SLIGHTLY SILTY
35				GRAY
			U-13	
	17		15 9 8 Std. 11 Pen 14	
40			U-15	
	36		18 16 Std. 20 Pen 28 16	Noted slight artesian at 45'
45				

SR 167 MP 13.77 to MP 14.73
15th St. S.W. to W. Main St. in Auburn
U-021-1(3) - 1971

Hole No. A-4 Sub Section East Bridge Pier #1 Sheet 3 of 5

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
			16 Std.	
	41		18 Pen	SANDY GRAVEL & GRAVELLY SAND, SLIGHTLY
			23	
			26 17	SILTY GRAY
50				
			U-18	
			16 Std.	
	36		17 Pen	
			19	
			17 19	
55				
			17 Std.	
	23		11 Pen	SAND - thinly scattered gravel, gray
			12	
			18 20	scattered wood & organic matter
60				
			4 Std.	
	18		9 Pen	
			9	
			9 21	
				SILT - Organic, brownish gray, layered with
65				extremely fine gray sandy silt & brown
			8 Std.	
	14		6 Pen	peat
			6	
	11		5 22	Sandy silt & silty sand - grey with organic
				matter & scattered gravels
0				

Form 351-003-a (H. F. 26.66-A).
Revised 5-67.

Hole No. A-4 Sub Section East Bridge Pier #1 Sheet 4 of 5

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
			U-23	69' gravelly silty sand - fine to
			11	coarse sand, gray, silty, gravels
	17		7 Std. 10 Pen 17 24	scattered throughout
75				
80				
	25		10 Std. 5 Pen 20 22 25	
85				
	19		6 Std. 7 Pen 12 7 26	
90				
	11		4 Std. 5 Pen 6 5 27	

167 MP 13.77 to MP 14.73
15th St. S.W. to W. Main St. in Auburn
U-021-1(3) - 1971

Log of Test Borings

Y Form 351-003-a (H. F. 26.65-A).
Revised 5-67.

Hole No. A-4 Sub Section East Bridge Pier #1 Sheet 5 of 5

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
			11 Std.	
	19		6 Pen	GRAVELLY SILTY SAND - fine to coarse sand gray
			13	
			13 28	silty, gravels scattered throughout
100				
			7 Std.	
	19		11 Pen	
			8	
			7 29	
				104' harder, appears to be same material
105				larger gravel & possible more gravel
			59 Std.	
	41		15 Pen	
			26	
			50 30	107' material, very dense, extremely hard
				driving casing, lost all water on top of
				hard layer at 107', 107'6" most all the water
110				back
			52 Std.	
	105/6"		105 Pen	107' Sandy gravel, silty binder, greenish gray
	50		26	
			24 31	Very dense, possible thin layers cemented
				No water lost in hole 0' to 107'
115				All samples damp to wet
			34	
	66		32	Logs &/or wood fragments possible throughout
			34 Std.	
			27 Pen	Std. Pen. 31 erratic blows, possible thin cemented
			22	
	47		20 32	layer
			27	
			38	
				STOPPED TEST BORING AT 119'-0"

HWY Form 351-003 (H. F. 26.66)
(Revised 5-671).

WASHINGTON
STATE HIGHWAY COMMISSION
DEPARTMENT OF HIGHWAYS

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LOG OF TEST BORING

S.H. ______ S.R. 167 Section West Auburn Interchange Pier #3 Job No. L-3598

Hole No. A-2 Sub Section SR 18 Over Crossing, East Bridge Cont. Sec. 176503

Station I^{m} 521+78 Offset 60' Right Ground El. 68.7'

Type of Boring Hollow flight augers & 3" casing Casing Augers 120'X10" Cas.141' W.T. El. 62.0'

Inspector ______ Date April 21, 1971 Sheet 1 of 6

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
				SANDY GRAVEL - Dense, gray, (Highway fill), fine to
				coarse sand, gravel and cobbles
				Logs or wood fragments possible through
				the depth of this boring
5				
			12 Std.	
			22 Pen	
	35		13 1	
			14	
10				SANDY GRAVELLY SILT - Slightly compact, gray, wet,
				scattered fine to coarse sand, gravel and
			9 Std.	cobbles
			9 Pen	SILT AND SAND - Slightly compact, gray, a marbled
	14		5 2	
			5	mixture, gray, moist silt, dark gray, wet
				fine sand
15				
			A B	
			C D	
			E U-3	
20			4	SILT WITH WOOD FRAGMENTS AND PEAT - slightly

SR 167 MP 13.77 to MP 14.73
15th St. S.W. to W. Main St. in Auburn
U-021-1(3) - 1971

Y Form 351-003-o (H. F. 26.66-A).
Revised 5-67.

Hole No. A-2 Sub Section SR 18 Over Crossing, East Bridge Sheet 2 of 6

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.		DESCRIPTION OF MATERIAL
	13		6 7 8		compact, gray, moist, a trace of very fine sand
			A B C E F G	U-5	
					FINE SAND - Compact to dense, dark gray a trace of silt
25	23		4 10 13 15	Std. Pen 6	
	43		14 22 21 28	Std. Pen 7	
30					SILTY, FINE SAND - Slightly compact, gray, peat and wood fragments scattered through
	18		7 8 10 12	Std. Pen 8	
35					Dense at about 36'
	43		10 13 30	Std. Pen 9	
40					
	40		15 22 18 20	Std. Pen 10	
45					

WY Form 351-003-a (H. F. 26.66-A).
Revised 5-67.

Hole No. A-2 Sub Section SR 18 Over Crossing, East Bridge Sheet 3 of 6

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
			14 Std.	SANDY GRAVEL - Compact, dark gray, clean,
			18 Pen	
	32		14 11	fine to coarse sand and fine to coarse gravel.
			14	
50				
				FINE SAND WITH WOOD FRAGMENTS - Compact, dark
			12 Std.	gray
			15 Pen	
	30		15 12	
			18	
55				GRAVELLY, SILTY, FINE SAND - Slightly compact,
				dark gray, peat and wood mixed through
			4 Std.	
			6 Pen	
	14		8 13	
			7	
60				
			3 Std.	
			6 Pen	
	19		13 14	SAND WITH SILT LENSES - Compact, dark gray
			16	fine sand with ½" ± lenses of gray
65				moist silt
			9 Std.	
			15 Pen	
	30		15 15	
			13	
70				SILTY, SANDY GRAVEL - Loose to slightly compact

SR 167 MP 13.77 to MP 14.73
15th St. S.W. TO W. Main St. in Auburn
U-021-1(3) - 1971

Log of Test Borings

Hole No. A-2 Sub Section SR 18 Over Crossing, East Bridge Sheet 4 of 6

	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	9		4 5 4 3 Std. Pen 16	gray, moist, fine to coarse sand, gravel and cobbles, wood fragments scattered through
75				
			A U-17	
	11		20 9 6 5 Std. Pen 18	
80				
	8		3 4 4 5 Std. Pen 19	
85				
			A B C D U-20	
90	13		6 4 9 11 Std. Pen 21	
9_				

WY Form 351-C03-a (H. F. 26.66-A).
Revised 5-67.

Hole No. A-2 Sub Section SR 18 Over Crossing, East Bridge Sheet 5 of 6

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
			8 Std.	
	16		7 Pen 9 22 26	
100				
105				
			8 Std.	Concentrated coarse gravel and cobbles
	24		10 Pen 14 23 16	106' to about 110'
110				
			6 Std.	
	20		7 Pen 13 24 18	
115				SANDY GRAVEL - Compact to dense, gray
				clean (heaves 20') fine to coarse sand
			17 Std.	gravel and cobbles
	62		30 Pen 32	
	25		13 25 12	

WY Form 351-003-a (H. F. 26.66-A).
Revised 5-67.

Hole No. A-2 Sub Section SR 18 Over Crossing, East Bridge Sheet 6 of 6

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	50/1"		50/1" Std. Pen 26	
125				
	51		12 17 34 Std. Pen 27	
130				
	51/6"		38 51 Std. Pen 28	
135				
	54/6"		34 54 Std. Pen 29	
140				SAND WITH SILT LENSES - Dense, dark gray fine sand with sea shells, slightly compact moist, gray silt
	31		5 10 21 27 34 Std. Pen 30	
				TEST BORING STOPPED AT 143'6"

R 167 MP 13.77 to MP 14.73
5th St. S.W. to W. Main St. in Auburn
-021-1(3) - 1971

Log of Test Borings

HWY Form 351-003 (H. F. 26.66)
(Revised 5-67).

WASHINGTON
STATE HIGHWAY COMMISSION
DEPARTMENT OF HIGHWAYS

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LOG OF TEST BORING

S.H. ______ S.R. 167 Section West Auburn Interchange Job No. L-3598

Hole No. H-2 Sub Section SR 18 O-Xing E. Br. Cont. Sec. 176503

Station LM 523+42 Pier #6 Offset 3' Lt. of ℄ Ground El. 64

Type of Boring Wash Bore (Blast) Casing 3" X 115' W.T. El. 63

Inspector ______ Date Nov. 13 to 19, 1969 Sheet 1 of 5

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
			B C U-1	SILT - brown, organic, damp
			D	PEAT - wood, organic matter, brown, damp
			3 4 Std. Pen	
	5		1 3 2	SAND - scattered silt lenses, wood
				gray, wet
5			U-3	
			3 3 Std. Pen	
	6		3 3 4	
10			C U-5	
			D E F	
			5 7 Std. Pen	
	15		8 8 6	
15			U-7	LOG
			4 4 Std. Pen	SAND - fine to coarse, scattered pieces
	13		9 22 8	Gravel, wood, & logs, gray, wet
20				

SR 167 MP 13.77 to MP 14.73
15th St. S.W. to W. Main St. in Auburn
U-021-1(3) - 1971

HWY Form 351-003-a (H. F. 26.66-A).
Revised 5-67.

Hole No. H-2 Sub Section ~~[illegible]~~ SR18 O-Xing E.Br Sheet 2 of 5

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	8		3 4 4 3 — Std. Pen 9	SAND - fine to coarse, scattered pieces Gravel, wood & logs, gray, wet
25			A B C D — U-10	
	3		2 1 4 11 — Std. Pen 11	
30			D E — U-12	SILTY SANDY GRAVEL - Gray, damp
	50		15 24 26 23 — Std. Pen 13	
35	35		18 14 21 13 — Std. Pen 14	SANDY GRAVEL - gray, wet appears water bearing
	25		11 12 13 16 — Std. Pen 15	
40	23		10 11 12 17 — Std. Pen 16	

SR 167 MP 13.77 to MP 14.73
15th St. S.W. to W. Main St. in Auburn
U-021-1(3) - 1971

Log of Test Borings

/Y Form 351-003-a (H. F. 26.66-A).
Revised 5-67.

Hole No. H-2 Sub Section ~~Ramp L U'Xing~~ SR 18 O-Xing E.Br. Sheet 3 of 5

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	18		11 9 9 8 Std. Pen 17	SILTY SAND - silt & sand, layered occasional piece gravel, gray, damp
50	25		10 13 12 15 Std. Pen 18	
55	20		6 8 12 30 Std. Pen 19	SAND - fine to medium, scattered coarse, trace gravel & silt, gray, wet
0	35		16 14 21 12 Std. Pen 20	SILTY SAND - gravelly, gray, trace wood, wet
5	19		7 6 13 9 Std. Pen 21	

HWY Form 351-003-a (H. F. 26.66-A).
Revised 5-67.

Hole No. H-2 Sub Section ~~Ramp L U'Xing~~ SR 18 O-Xing E.Br. Sheet 4 of 5

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	7		6 3 4 4 Std. Pen 22	SILTY SAND - Gravelly, gray, trace wood, wet
	8		7 4 4 6 Std. Pen 23	
75				
80				
	8		5 4 4 7 Std. Pen 24	
85				
	8		3 3 5 8 Std. Pen 25	
	16		3 3 13 6 Std. Pen 26	
90				

WY Form 351-003-a (H. F. 26.66-A)
Revised 5-67.

Hole No. H-2 Sub Section ~~Ramp I-UXing~~ SR18 O-Xing E.Br. Sheet 5 of 5

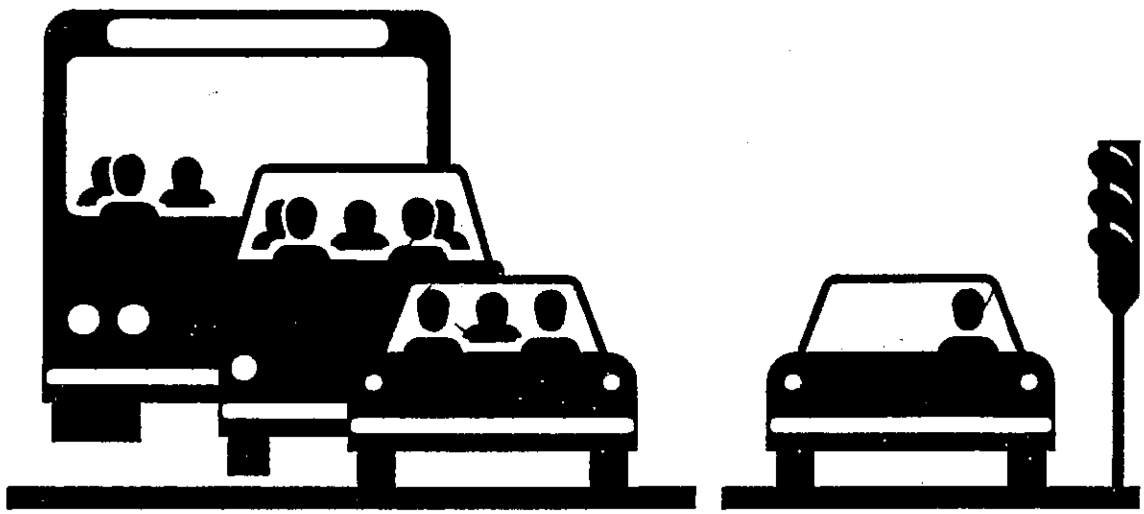
DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	18		12 9 9 13 — 27	SILTY SAND - gravelly gray trace wood, wet
100	17	100	17 10 7 11 — 28	
				GRAVEL - sandy, gray, silt binder, occasional cobble & boulder, damp
105	104		21 61 43 92 — 29	
				108'6" boulder - 109' blast 6 sticks 50% 8" X 1 1/8"
110	54		35 25 29 26 — 30	
115	60	115	62 34 26 33 — 31	
				STOPPED TEST BORING AT 117'-0"
120				

SR 167 MP 13.77 to MP 14.73
15th St. S.W. to W. Main St. in Auburn
U-021-1(3) - 1971

SR-167

15TH STREET S.W. TO SOUTH GRADY WAY

Bridge Foundation Report



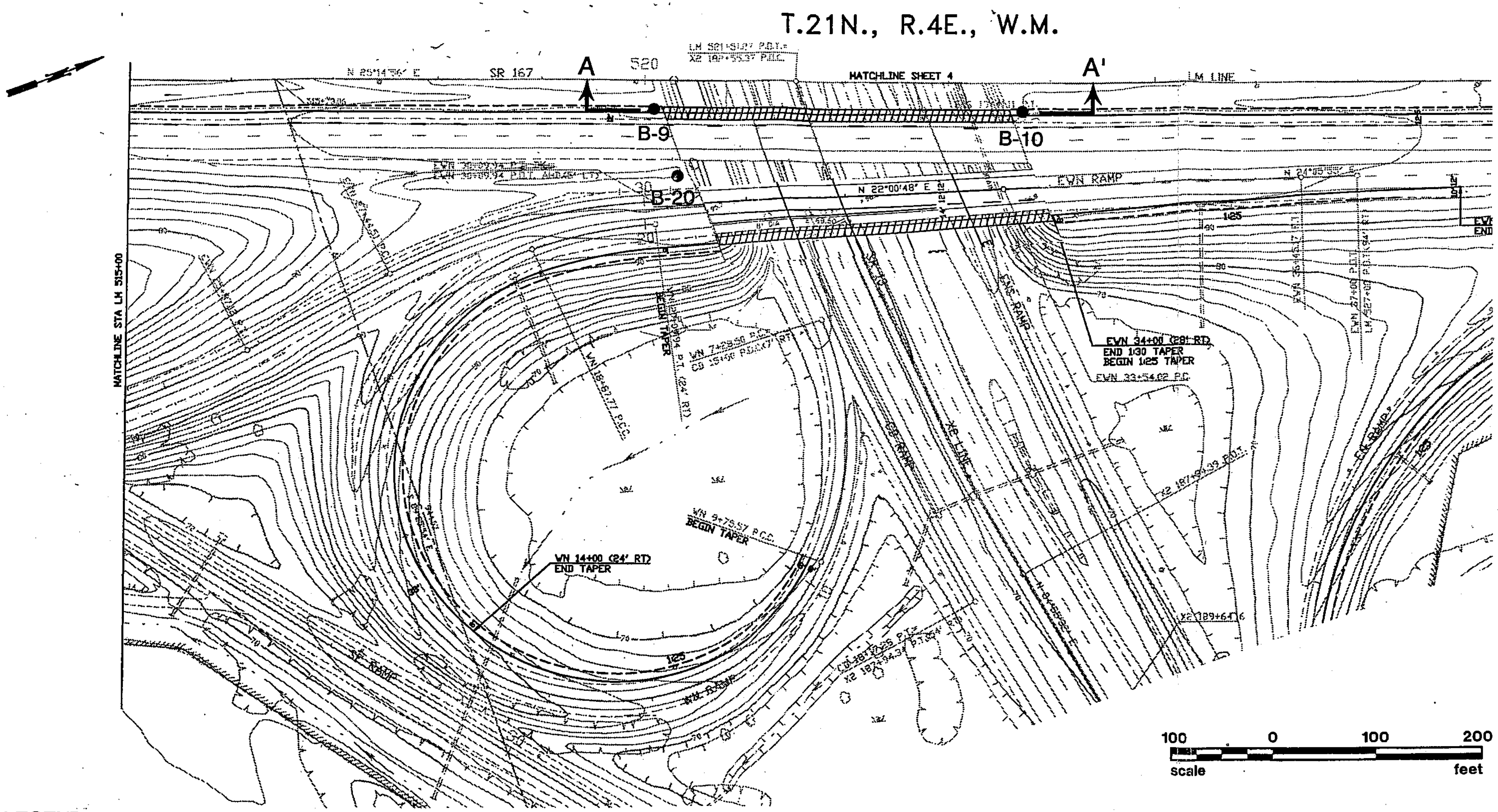
◊ HOV Improvements

◊ Ramp Metering

◊ Surveillance, Control, and Driver Information

Washington State
Department of Transportation

TERRA ASSOCIATES, Inc.
Consultants in Geotechnical Engineering, Geology
and
Environmental Earth Sciences



LEGEND

● B-1 APPROXIMATE TEST BORING LOCATION

REF: Based on WSDOT SR-167, 15th Street SW to South Grady Way Alternative Analysis.

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EXPLORATION LOCATION PLAN
STATE ROUTE 167
STATE ROUTE 18 BRIDGE

Proj. No. 1630	Date 10-91	Figure 2

BORING NO. 9

Logged By DBG

Date 6-18-91

ELEV. 96±

Graph	US CS	Soil Description	Depth (ft.)	Sample	(N) Blows Ft.	W (%)
	SM	Gray-brown, very silty, gravelly SAND, fine to medium grained, moist, medium dense. (Fill)	5		26	15
					22	14
			10			
		Very silty.			100	9
			15			
					43	17
			20			
		Medium dense.			21	18
			25			
					15	11
			30			
					62	7
		Becomes dense.	35			
	P_T	Brown Peat, wet, soft.			3	277
			40			
	ML	Gray sandy SILT, wet, loose.			9	44
		Occ'l sand layers; SILT with clay	45			
					29	24
			50			
	SM	Black silty SAND, fine grained, wet, medium dense.			31	33
			55			
					6	22
	SP	Becomes slighty silty, SAND with silt.	60			
					15	26
			65			

BORING CONTINUED ON NEXT PAGE

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BORING LOG

STATE ROUTE 167

KING COUNTY, WASHINGTON

Proj. No. 1630 | Date 10-91 | Figure A-10

BORING NO. 9

Logged By DBG
Date 6-18-91

ELEV. 96±

Graph	USCS	Soil Description	Depth (ft.)	Sample	(N) Blows Ft.	W (%)
	SW	Black gravelly SAND, wet, dense		I	33	16
			70			
		Becomes more gravelly		I	100	15
			75			
				I	54	14
			80			
				I	63	13
			85			
				I	51	6
			90			
				I	84	*
			95			
	SM	Black, very silty SAND		I	51	47
			100			
	ML	Gray, slighty gravelly, fine to coarse sandy SILT, wet, medium dense.		I	18	13
			105			
				I	26	15
			110			
				I	22	15
			115			
		Some wood.		I	23	*

Boring completed at depth 119 feet.
Groundwater noted at 37 feet.

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BORING LOG
STATE ROUTE 167
KING COUNTY, WASHINGTON

Proj. No. 1630 | Date 10-91 | Figure A-10

BORING NO. 10

Logged By DBG
Date 6-20-91

ELEV. 96±

Graph	USCS	Soil Description	Depth (ft.)	Sample	(N) Blows Ft.	W (%)
	SM	Tan, silty, gravelly SAND, fine to medium grained, moist, dense. (Fill)	5		100	10
			10		71	9
			15		52	14
		Becomes gray-tan.	20		100	11
			25		53	13
			30		56	14
			35		83	10
			40		90+	*
	ML	Gray sandy SILT, wet, soft.	45		12	33
	SM	Black silty SAND, fine grained, wet, medium dense.	50		48	23
		Some gravelly SAND with silt.				
	SW	Black SAND, fine to coarse grained, wet, loose.	55		8	38
		Wood entire sample.	60		100+	75
		Becomes gravelly.	65		14	19

BORING CONTINUED ON NEXT PAGE

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BORING LOG
STATE ROUTE 167
KING COUNTY, WASHINGTON

Proj. No. 1630 | Date 10-91 | Figure A-11

BORING NO. 10

Logged By DBG

Date 6-20-91

ELEV. - 96±

Graph	USCS	Soil Description	Depth (ft.)	Sample	(N) Blows Ft.	W (%)
				I	49	8
			70			
	SM	Becomes silty and dense.		I	95+	21
			75			
				I	86	19
			80			
				I	44	*
			85			
		Becomes loose, with extensive organic matter.		I	11	50
			90			
	ML	Gray fine sandy SILT, wet, hard.		I	42	26
			95			
				I	50	23
			100			
				I	18	15
			105			
				I	16	16
			110			
				I	15	17

Boring completed at depth 114 feet.
Groundwater noted at 44 feet.

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BORING LOG
STATE ROUTE 167
KING COUNTY, WASHINGTON

Proj. No. 1630	Date 10-91	Figure A-11

BORING NO. 20

Logged By DBG

Date 7-8-91

ELEV. 96±

Graph	USCS	Soil Description	Depth (ft.)	Sample	(N) Blows Ft.	W (%)
	SM	Gray, slightly gravelly, silty SAND, fine to medium grained, moist, dense. (FILL)		I	36	17
			5			
				I	29	12
			10			
				I	51	13
			15			
				I	40	15
			20			
				I	25	22
			25			
				I	22	16
			30			
				I	94+	14
	GW	Grades to fine to coarse sandy gravel, wet. (FILL)	35			
				I	27	12
			40			
	ML	Gray, slightly sandy SILT, wet, stiff.		I	12	38
			45			
	SM	Black, silty SAND, fine to medium grained, wet, loose.		I	13	23
			50			
				I	6	46
	SP	Clean SAND lenses.	55			
				I	11	23
			60			
				I	62	9
		Becomes fine to coarse grained and dense.	65			

BORING CONTINUED ON NEXT PAGE

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BORING LOG

STATE ROUTE 167

KING COUNTY, WASHINGTON

Proj. No. 1630 | Date 10-91 | Figure A-21

BORING NO. 20

Logged By DBG
Date 7-8-91

ELEV. 96±

Graph	US CS	Soil Description	Depth (ft.)	Sample	(N) Blows Ft.	W (%)
	SM	Black, silty, gravelly SAND, fine to coarse grained, wet, dense.	70	I	94	20
				I	23	3
		Becomes very gravelly.	75	I	100+	15
			80	I	100+	5
			85	I	55	13
			90	I	100+	13
			95			
	SW	Becomes slightly silty		I	69	10

Boring completed at depth 99 feet.
Groundwater noted at 33 feet.

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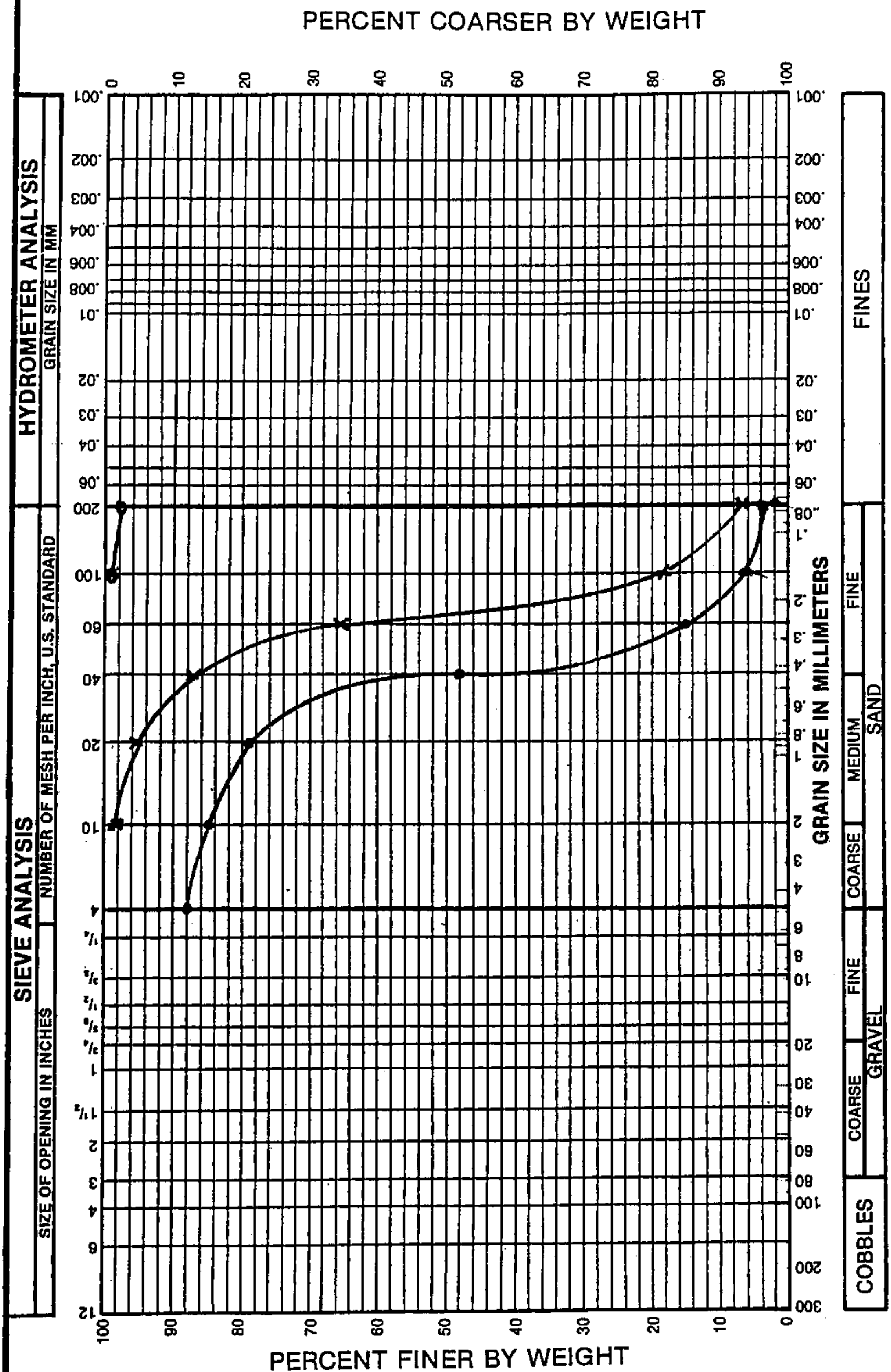
BORING LOG
STATE ROUTE 167
KING COUNTY, WASHINGTON

Proj. No. 1630 | Date 10-91 | Figure A-21



Key	Boring or Test Pit	Depth (ft.)	USCS	Description	Moisture Content (%)	LL	PL
✱	B-8	33	SP/SM	SAND with silt	21		
●	B-8	48	SP	SAND	30		
○	B-9	43	ML	SILT with clay	44		

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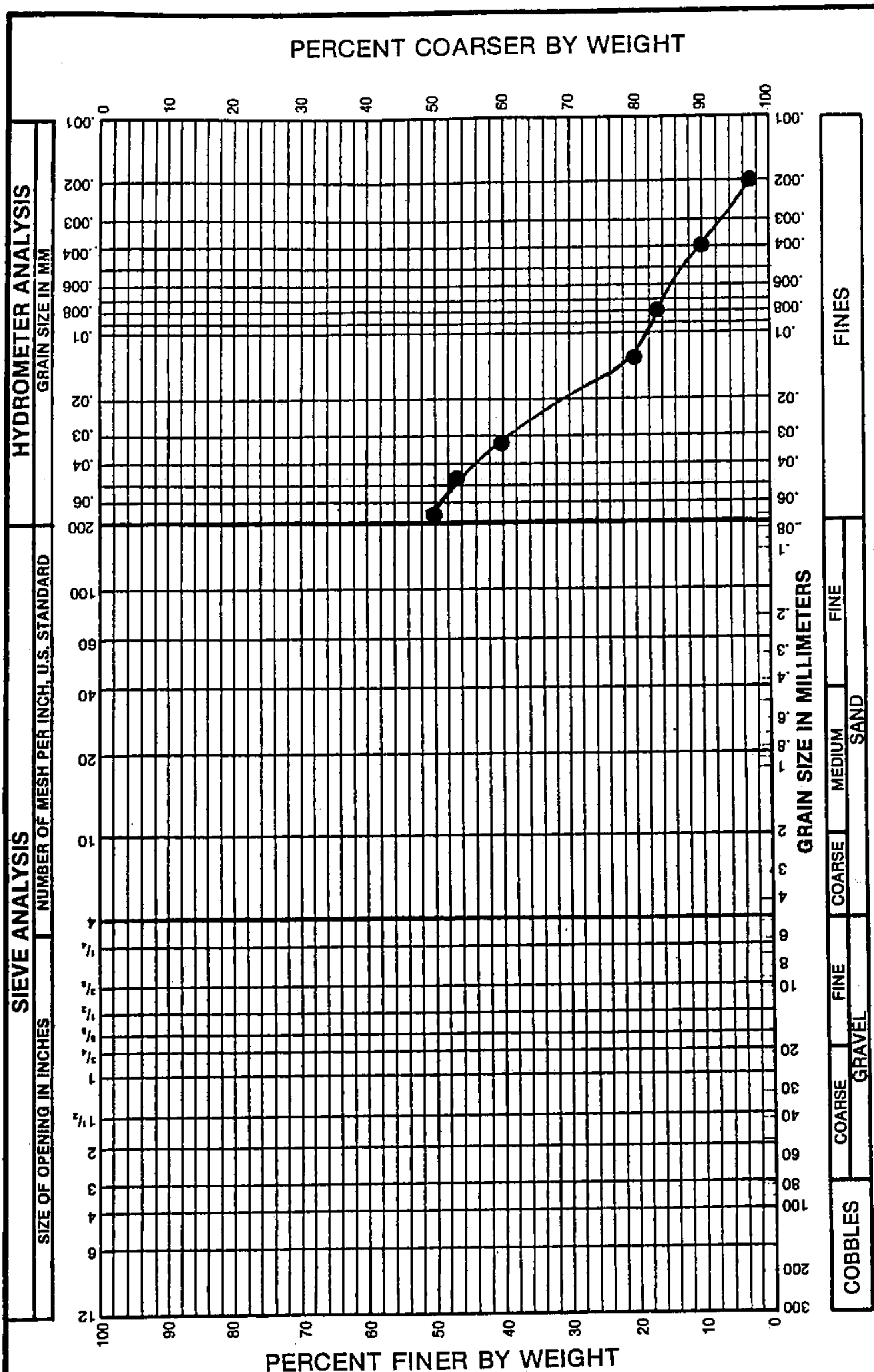
GRAIN SIZE ANALYSIS

State Route 167
King County, Washington

Proj. No. 1630 | Date 10-91 | Figure B-8



Key	Boring or Test Pit	Depth (ft.)	USCS	Description	Moisture Content (%)	LL	PL
●	B-9	42.5	ML	SILT with sand and clay	43.0	-	-

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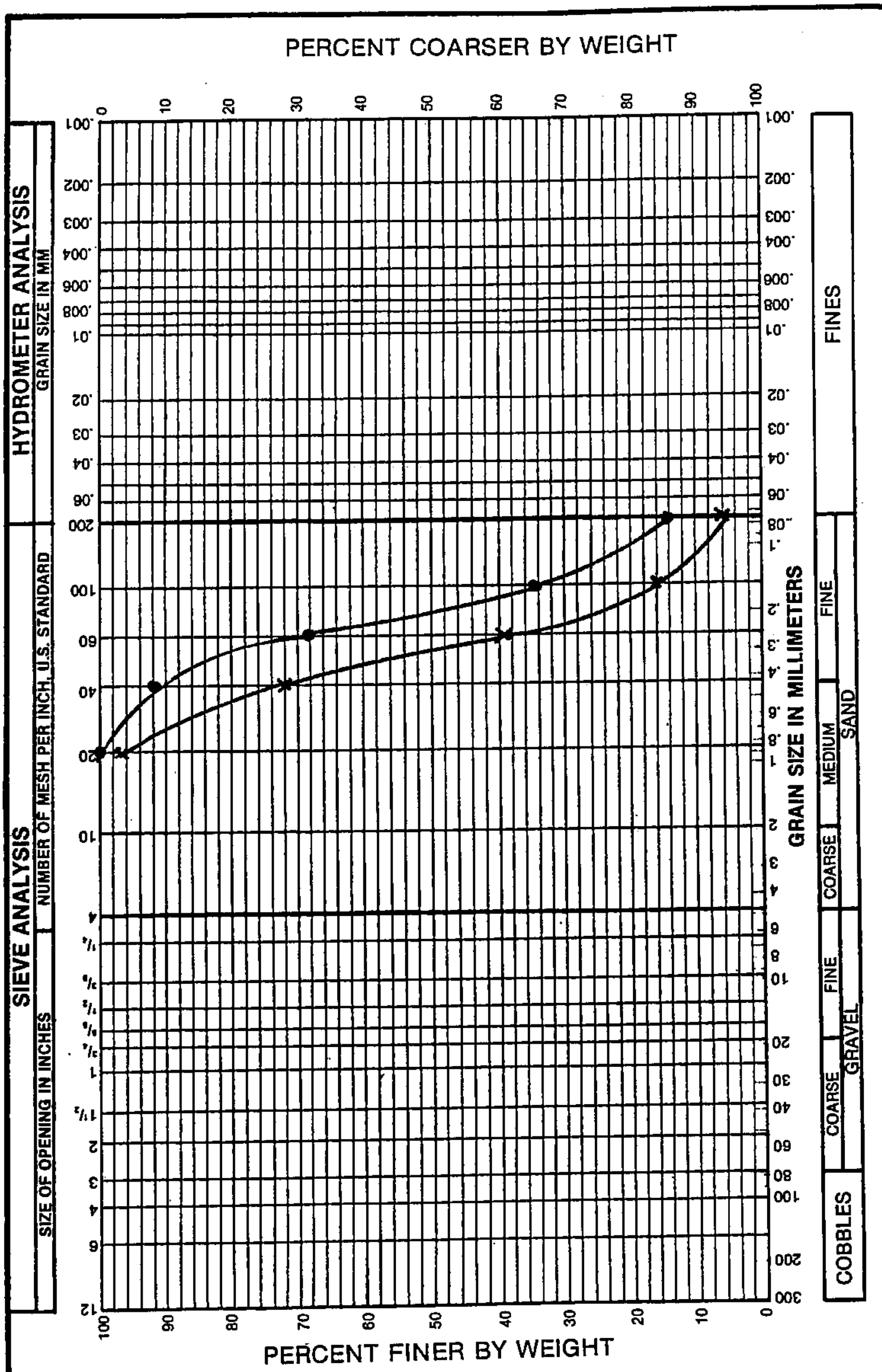
GRAIN SIZE ANALYSIS

State Route 167
King County, Washington

Proj. No. 1630	Date 10-91	Figure B-9



Key	Boring or Test Pit.	Depth (ft.)	USCS	Description	Moisture Content (%)	LL	PL
—×—	B-9	58	SP	SAND with silt	22		
—●—	B-9	63	SM	silty SAND	26		

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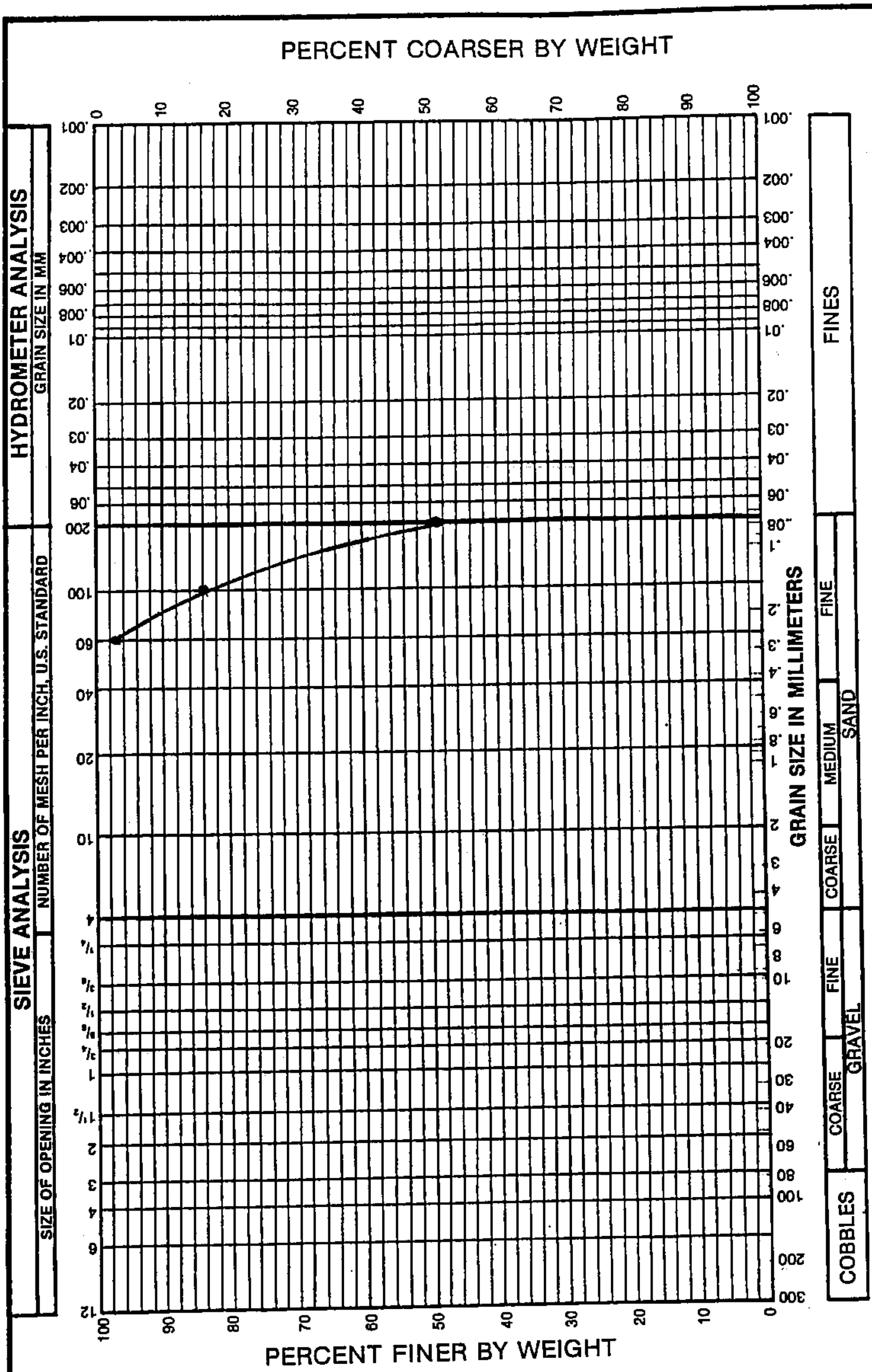
GRAIN SIZE ANALYSIS

State Route 167
King County, Washington

Proj. No. 1630	Date 10-91	Figure B-10



Key	Boring or Test Pit	Depth (ft.)	USCS	Description	Moisture Content (%)	LL	PL
	B-10	43	SM/ML	silty SAND to sandy SILT	33		

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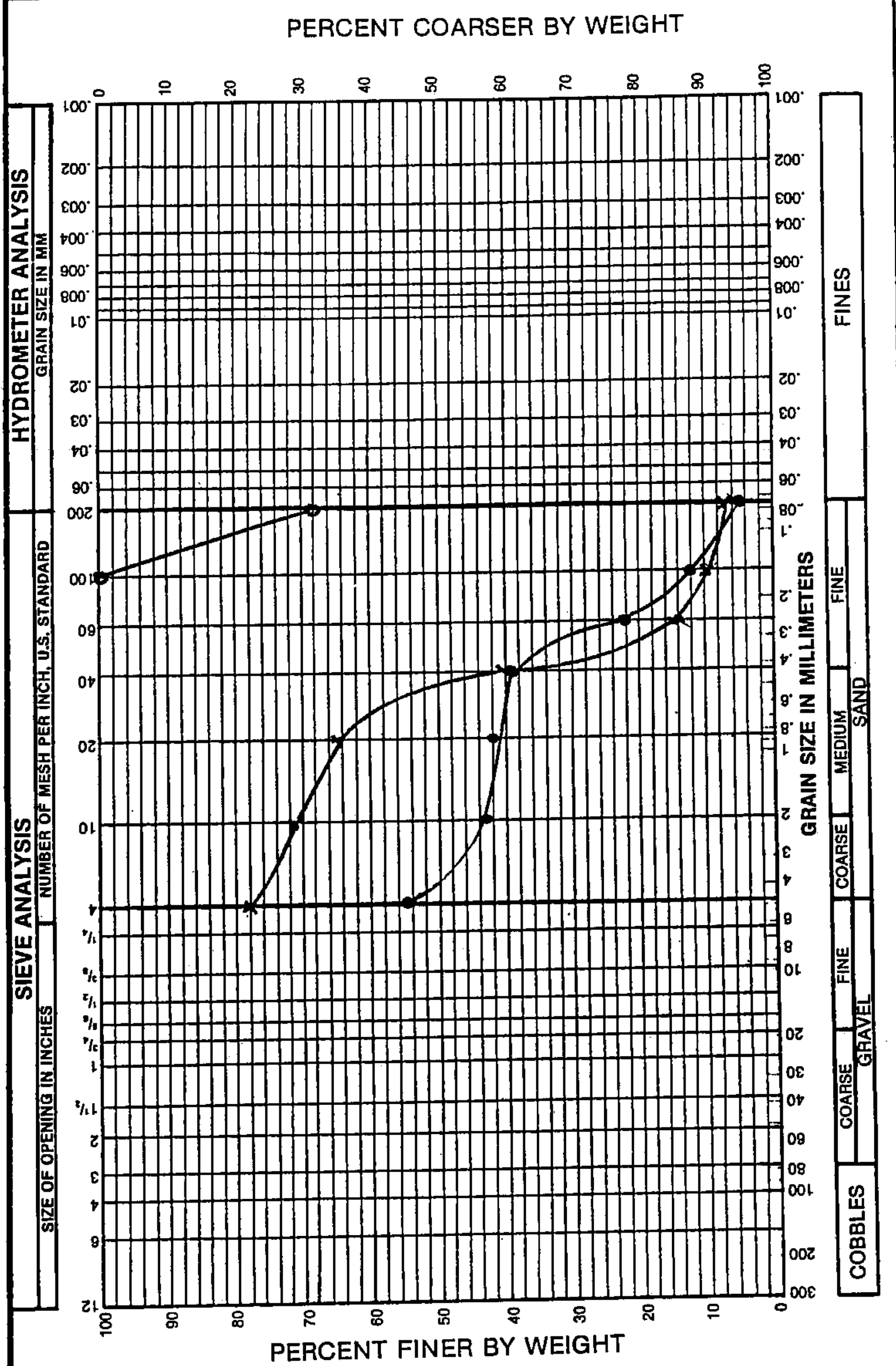
GRAIN SIZE ANALYSIS

State Route 167
King County, Washington

Proj. No. 1630	Date 10-91	Figure B-11



Key	Boring or Test Pit	Depth (ft.)	USCS	Description	Moisture Content (%)	LL	PL
—x—	B-10	53	SM/SP	gravelly SAND with silty with wood	38		
—•—	B-10	63	SM/SP	gravelly SAND with silt	19		
—o—	B-11	23	ML	sandy SILT	31		

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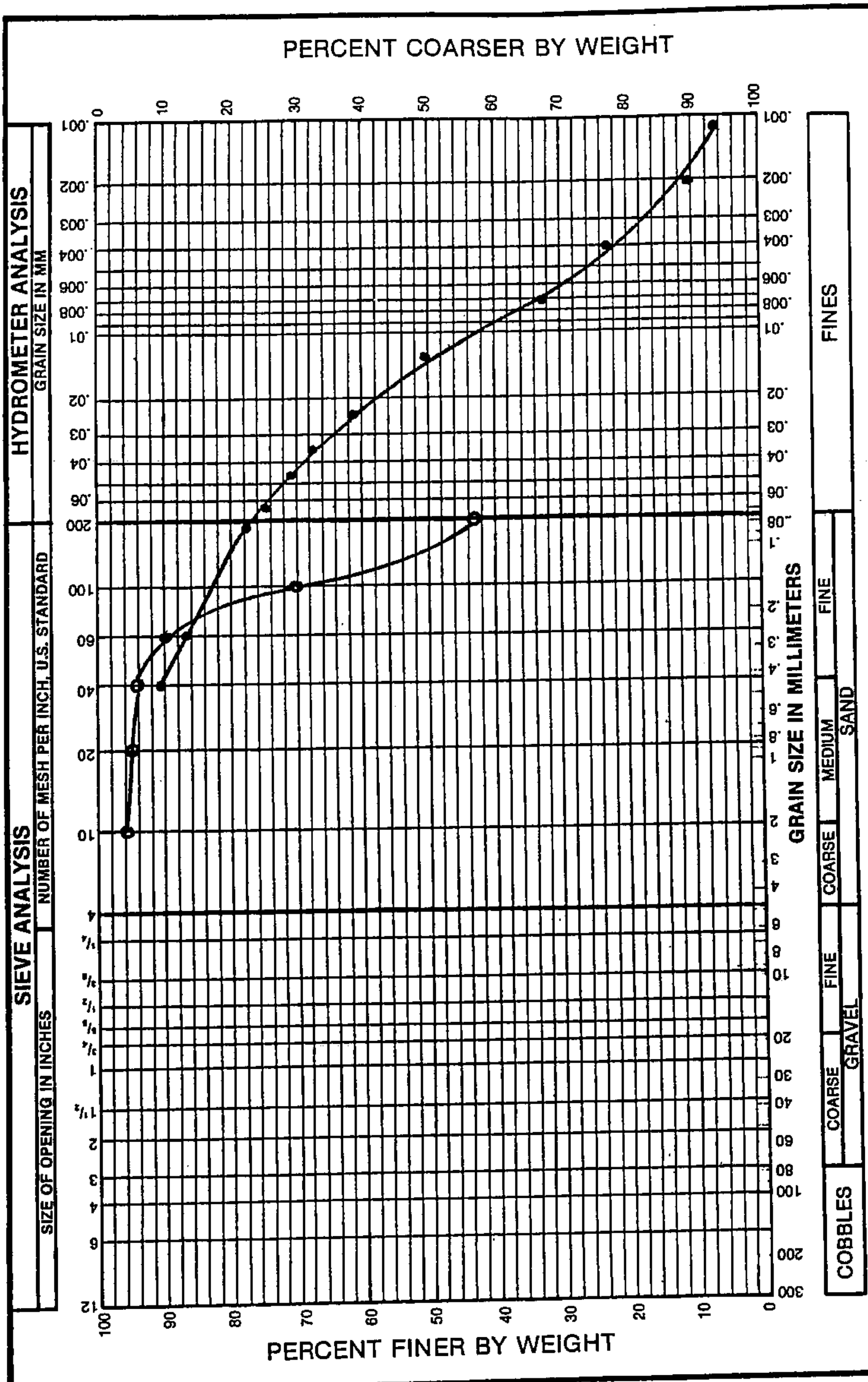
GRAIN SIZE ANALYSIS

State Route 167
King County, Washington

Proj. No. 1630 | Date 10–91 | Figure B–12



Key	Boring or Test Pit	Depth (ft.)	USCS	Description	Moisture Content (%)	LL	PL
—●—	B-19	38	ML	sandy SILT	36		
—o—	B-20	53	SM	silty SAND	46		

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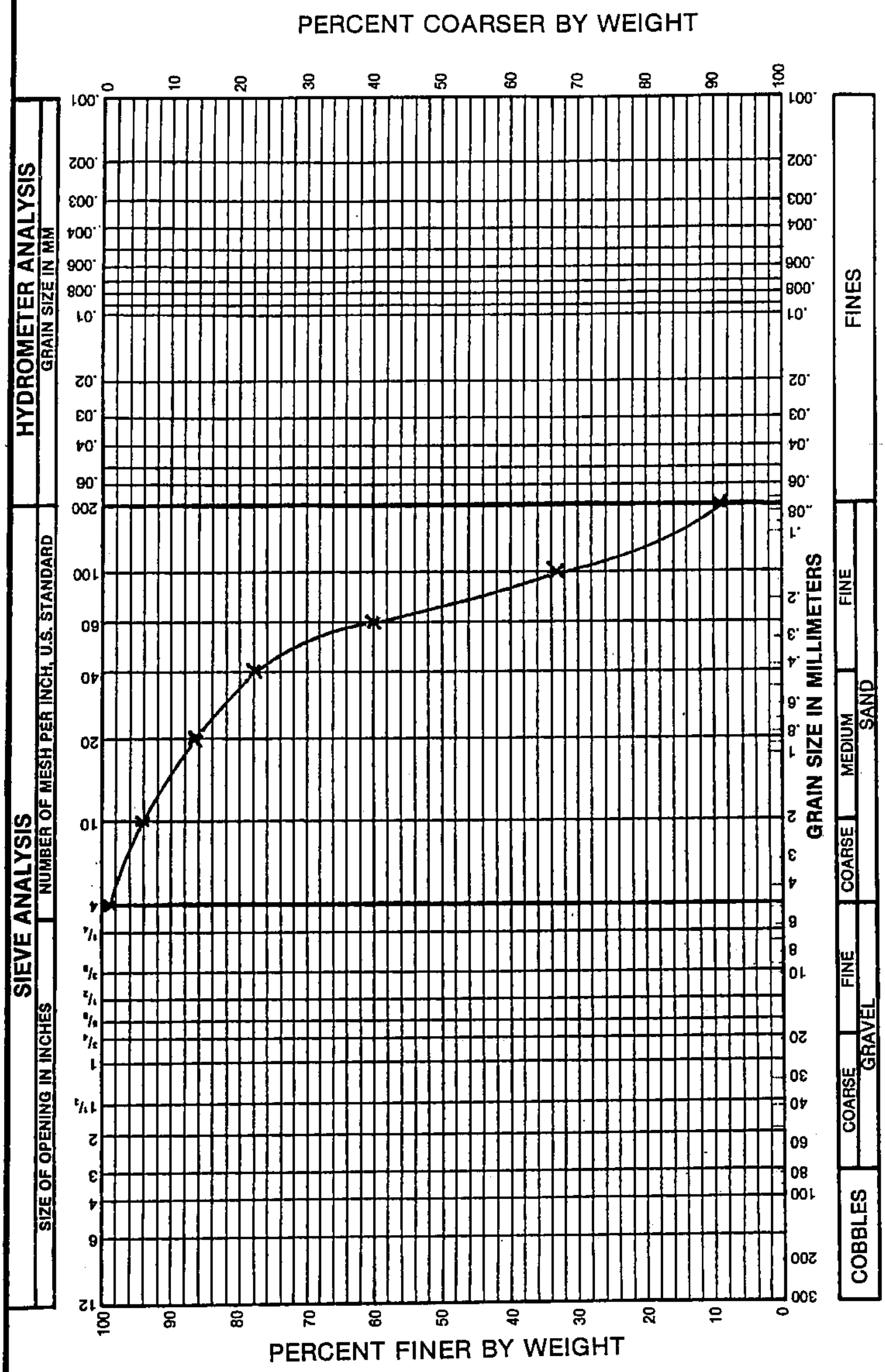
GRAIN SIZE ANALYSIS

State Route 167
King County, Washington

Proj. No. 1630 | Date 10-91 | Figure B-21



Key	Boring or Test Pit	Depth (ft.)	USCS	Description	Moisture Content (%)	LL	PL
—✕—	B-20	58	SP/SM	SAND with silt	23		

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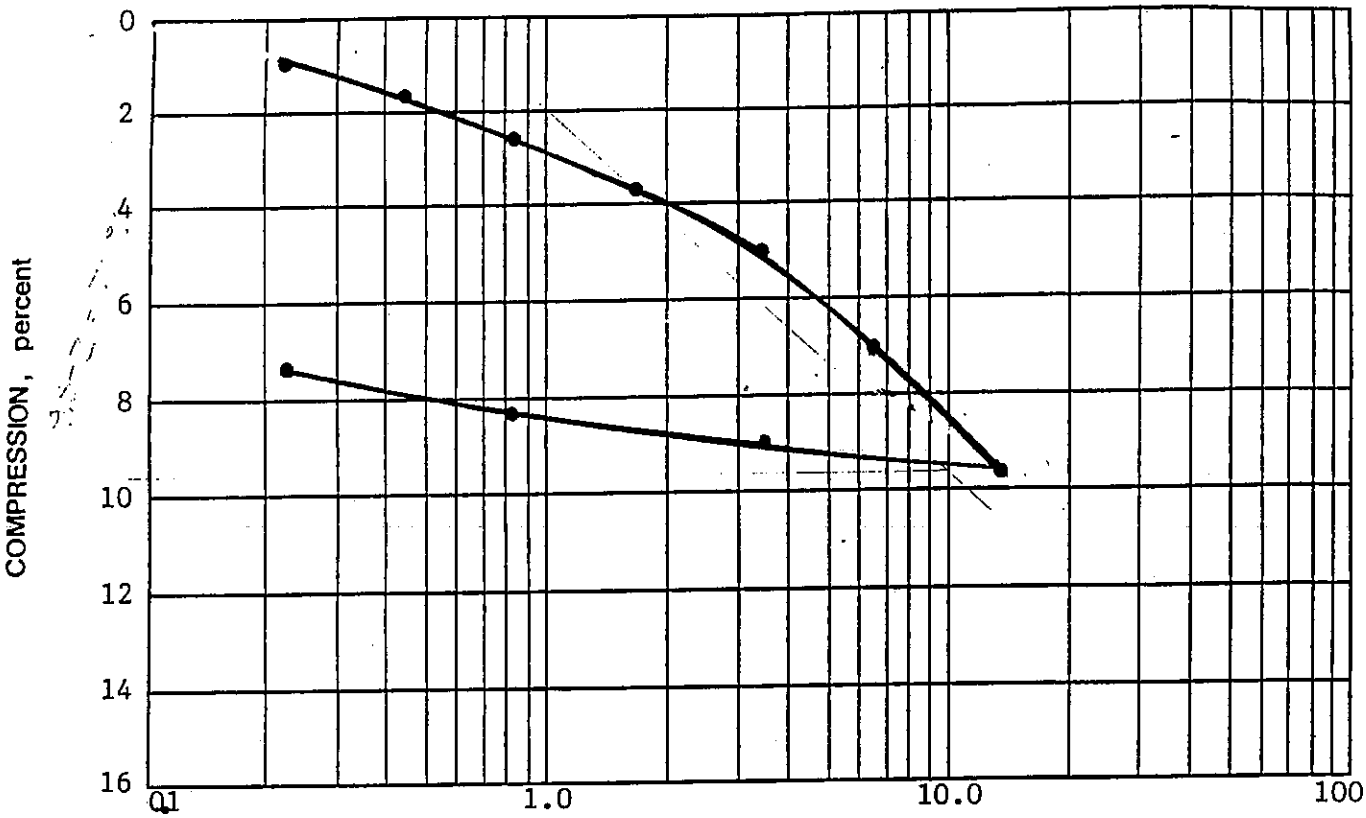
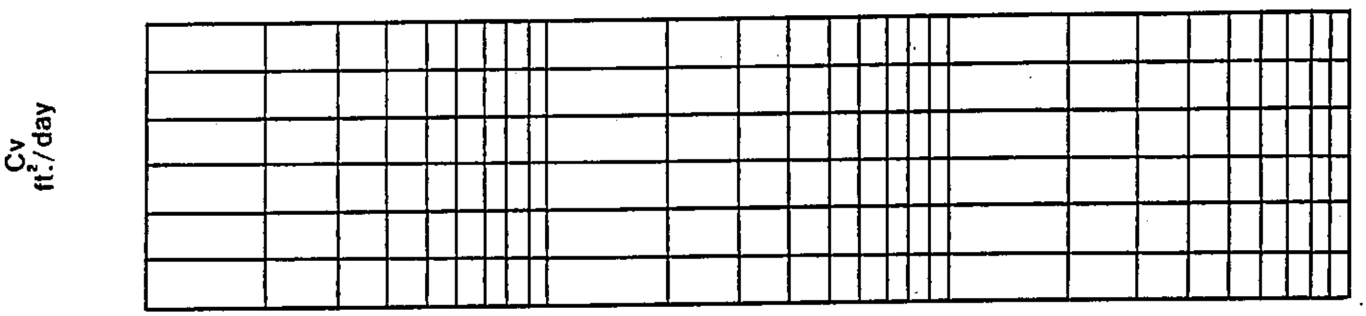
GRAIN SIZE ANALYSIS

State Route 167
King County, Washington

Proj. No. 1630 | Date 10-91 | Figure B-22





PRESSURE, tsf

Key	Boring No.	Depth (ft.)	USCS	Soil Description	Liquid Limit %	Plastic Limit %	Plasticity Index %	Moisture Content, W% Before	Moisture Content, W% After	Dry Density (pcf)
—●—	B-10	42.5	ML	Sandy SILT with clay	–	–	–	31.3	26.7	89

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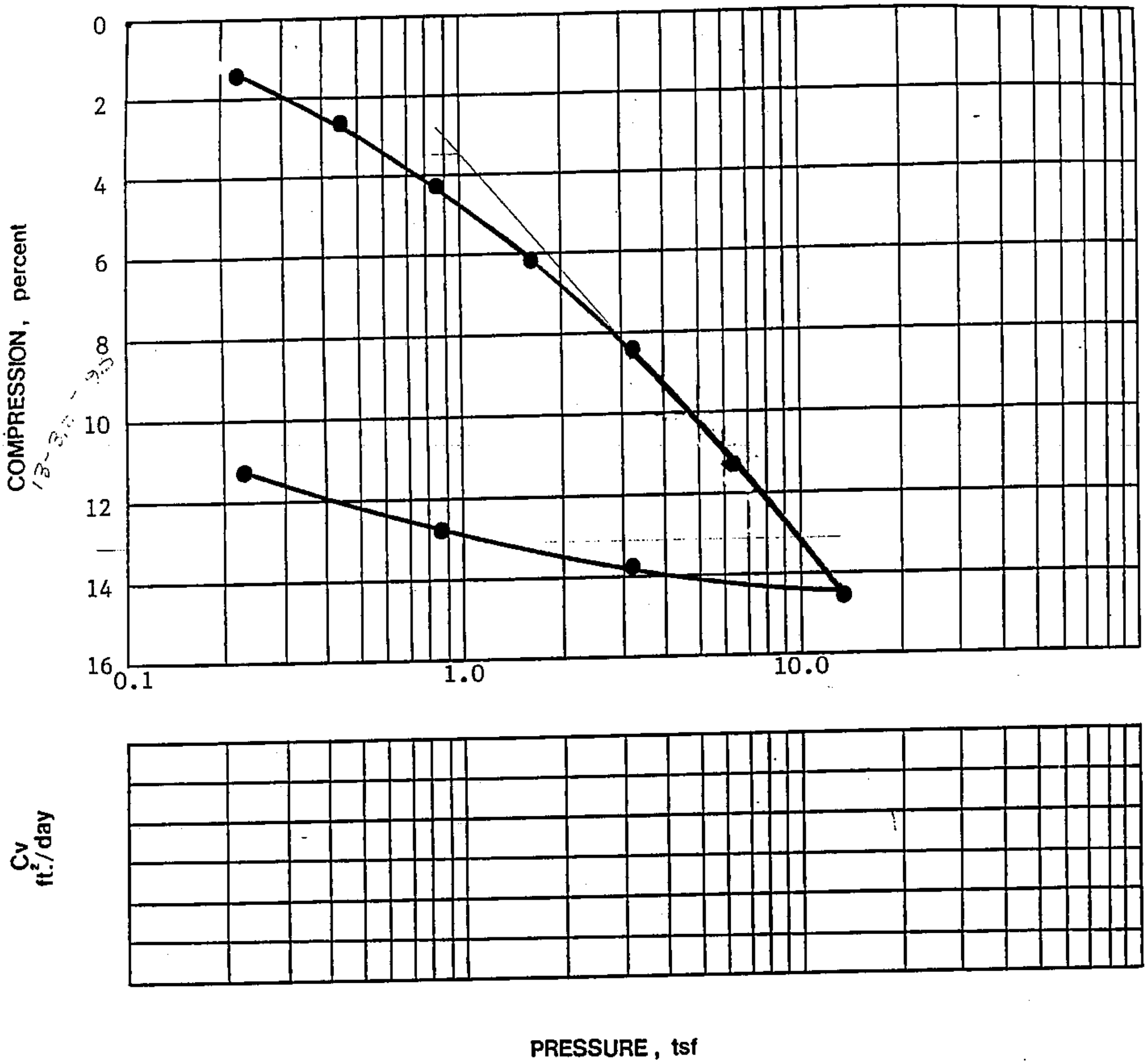
CONSOLIDATION TEST DATA

State Route 167 Bridges
Kent/Auburn, Wash.

Proj. No. 1630 | **Date** 10-91 | **Figure** B-23



Key	Boring No.	Depth (ft.)	USCS	Soil Description	Liquid Limit %	Plastic Limit %	Plasticity Index %	Moisture Content, W% Before	Moisture Content, W% After	Dry Density (pcf)
—●—	B-9	42.5	ML	Sandy SILT, with clay	–	–	–	43.0	36.0	73

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CONSOLIDATION TEST DATA
State Route 167 Bridges
Kent/Auburn, Wash.

Proj. No. 1630 | **Date** 10-91 | **Figure** B-25

Chapter 6 -- Bridge No. 167/112 W-N Ramp

Foundation design studies carried out for Bridge No. 167/112 W-N Ramp (W-N Ramp) included

- determining the axial capacity of driven piles and drilled shafts
- assigning soil properties for use in lateral response analyses of driven piles and drilled shafts, and
- estimating the allowable bearing pressures for the abutment footings.

In view of the potential for liquefaction of sands and silts prevalent in the upper 12 m (40 ft) of soil profile at the bridge site, the possible effects of liquefaction on the axial and lateral capacity of driven piles and drilled shafts, as well as the stability of abutment slopes, were also evaluated. Methods used during and key results from these foundation capacity and liquefaction analyses are presented in this chapter.

Project Design Considerations

The W-N Ramp structure is located between 15th Avenue SW and 15th Avenue NW, where SR-167 crosses over SR-18. The general location of the bridge is shown in Figure 1-1. This bridge will be widened on its east side by 1.5 to 3.7 m (5 to 12 ft) to provide an HOV lane.

Existing Structure

The W-N Ramp structure was constructed in the early 1970's from prestressed concrete. It is approximately 101 m (331 ft) in length, and its width varies from approximately 14 m (46 ft) at its south end to 12 m (40 ft) at the north end. The bridge is supported on four interior piers with each pier consisting of two columns. Columns have an exposed height of approximately 5.8 m (19 ft). Interior piers of the bridge are located approximately 18 to 27 m (58 to 90 ft) apart. The ends of the bridge are supported by shallow strip footings located within the abutment fill.

The foundation for each column consists of a pile cap located approximately 1 to 2 m (3 to 7 ft) below the roadway surface. From the original design drawings, it appears that each pile cap is roughly 3 m by 3 m (10 ft by 10 ft) in plan and is located approximately 7.5 m (25 ft) from the adjacent pile cap within the pier. Each pile cap is supported by six driven concrete piles. The estimated average length of the concrete piles, based on the original design drawings, is 12 m (40 ft). This results in the toe of the piles being located at an approximate elevation of 5 to 9 m (16 to 30 ft). The concrete piles were required in the design drawings for the bridge to have a capacity of 490 kN (55 ton) .

Approach fills for the bridge are approximately 8 m (26 ft) in height. The end of the abutment fill is sloped at 2H:1V (horizontal to vertical); side slopes on the east side of the approach fill range in steepness from 3H:1V to 2.5H:1V. A 1.5-m (5 ft) wide strip footing is located at each end of the bridge in the approach fill, approximately 3 m (10 ft) below the roadway surface. Design drawings indicate that the allowable bearing pressure on the footing is 290 kPa (3 tsf).

Site Conditions

The site is level except for the grade change to accommodate the approach fills for the bridge. Areas along the east side of the bridge abutments, where widening will occur, are covered with grasses, brush, and small trees. These areas should pose no significant obstructions to construction.

Traffic on the W-N Ramp bridge and SR-18 are heavy and will present significant construction constraints. Median widths for Piers 3 and 4 are approximately 5 m (16 ft); Piers 2 and 5 are located at the toe of the approach fill.

Subsurface Conditions

Seven test holes have been drilled and sampled for this bridge: three for the original bridge design and four as part of this task order. A piezometer was installed in one of the test holes completed for this task order. Locations of the test holes are shown in Figure 6-1. Test hole logs based on past and the most recent explorations are included at the end of this report chapter. Limited numbers of laboratory grain-size tests were also completed as part of this task order. Results of these tests are also included at the end of this chapter.

The geotechnical soil profile for this bridge consists of layered silts, sands, and gravels to the maximum depth of exploration, 38 m (125 ft). Figure 6-2 shows the soil profile that was developed from the test hole logs.

For the purposes of the foundation design studies, six primary soil layers are identified. The characteristics and approximate depths of these layers are summarized as follows, beginning at the ground surface:

- **Layer 1 -- Site Fill:** This material occurs from the ground surface to approximate elevation 17 to 18 (56 to 60 ft). It appears that approximately 3 m (10 ft) of the site soil were removed during original construction and were replaced with this material. The same material is used for the approach fills to the bridge. Generally the fill is a dense sandy gravel. From location to location and depth to depth, the amount of silt changes. This layer is generally above the water table; blowcounts from the SPT are normally greater than 20.

- **Layer 2 – Sandy Silt Layer:** This layer extends from approximate elevation 17 (56 ft) to approximate elevation 14 m (46 ft) near the southern piers (Piers 2 and 3) and from approximate elevation 18 m (60 ft) to elevation 15 m (49) near the northern piers (Piers 4 and 5). The material is primarily fine silty sand and sandy silt. Blowcounts are often less than 10. It is located below the water table.

- **Layer 3 -- Sand and Gravel Layer:** This layer occurs between approximate elevation 14 m (46 ft) and elevation 1 m (3 ft) near the southern piers and from approximate elevation 15 m (49 ft) to elevation 5 m (17 ft) near the northern piers. The layer consists of a gravelly sand to sandy gravel with some wood debris near the northern pier locations. Blowcounts near the southern piers are often above 25; the blowcounts near the northern piers can be less than 25 with some, near the top and bottom of the layer less than 10.

- **Layer 4 – Sandy Silt Layer:** This layer occurs between elevation 1 m (3 ft) and elevation –2 m (-7 ft) near the southern piers and between elevation 5 m (17 ft) and elevation 0 m (0 ft) near the northern piers. The layer consists generally of silt and sand with some organics. Some clay is also present near the southern piers. Blowcounts in this layer can be as low as 10.
- **Layer 5 -- Loose Sand Layer:** This layer consists of nearly 15 m (49 ft) of loose silty sand and gravelly sand with some silt. Traces of wood are noted in the test hole logs. Blowcounts range from 5 to 20 or more. Blowcounts in the top 5 m (16 ft) of this layer are often less than 10. Higher blowcounts occur at deeper depths.
- **Layer 6 -- Dense Gravel Layer:** A dense gravel layer is encountered at approximate elevations -13 to -15 m (-43 to -49 ft). This layer is very consistent in the general area. Blowcounts from the SPT are in excess of 50 blows per 0.3 m (1 ft).

Several important features within the soil profile were identified from the test hole logs. First, low blowcounts occur within Layers 2, 3, 4, and the upper portion of Layer 5. While some of these low blowcounts appear to be caused by heave within the augers during drilling, at least some are thought to represent actual conditions. As discussed subsequently, the low blowcounts in Layers 2 and 3 lead to concerns about the susceptibility of these layers to liquefaction during a design earthquake. The low blowcounts in Layer 4 and especially the top portion of Layer 5 present concerns about the depths at which end bearing can be mobilized in driven piles or drilled shafts.

Another relevant observation during both the present and past exploration programs was the presence of scattered wood fragments and cobbles within the soil profile A boulder that required blasting with dynamite was encountered at a depth of 34 m (112 ft) in another test hole (H-3-69).

Groundwater was measured at depths of 1.5 to 3 m (5 to 10 ft) below the ground surface. These depths correspond to approximate elevations of 18 to 20 m (60 to 66 ft). Slight artesian conditions were also reported at a depth of 11 m (36 ft) during drilling of one test hole (A-5-71). A design groundwater elevation of 20 m (66 ft) was used for static pile and drilled shaft analyses. For liquefaction analyses the groundwater elevation was assumed to be elevation 18 m (60 ft). This lower level for liquefaction analyses represented an expected long-term condition, while the higher elevation was used for pile and drilled shaft design to assure that adequate conservatism was incorporated in design for possible short-term loading conditions.

Engineering Soil Properties

Engineering properties were assigned for each of the primary soil layers to aid in subsequent foundation design computations. Various methods were used to assign these properties, including soil descriptions, blowcounts from the SPTs, and normal engineering judgment. These properties are best-estimated values, rather than lower bound. The fact that the values are best-estimates needs to be recognized as factors of safety are selected for determining the axial capacity of driven piles and drilled shafts. Summaries of these properties are presented in Table 6-1.

Table 6-1. Summary of Estimated Soil Properties at W-N Ramp

Soil Layer No.	Moist Unit Weight (kN/m^3)	Saturated Unit Weight (kN/m^3)	Friction Angle	
			Piers 2 & 3	Piers 4 & 5
1	19.6	-	33	33
2	-	18.1	29	29
3	-	19.6	33	30
4	-	18.9	30	30
5	-	18.9	30	30
6	-	20.3	35	35

Liquefaction Susceptibility

Liquefaction assessments were conducted using the Seed-Idriss simplified blowcount procedure (Seed and Idriss, 1982) with a peak ground acceleration of 0.35g. As noted in Chapter 3 of this report, the peak firm-ground acceleration for the site is estimated to be 0.29g. This motion is expected to amplify by a factor of approximately 1.2, as the seismic wave propagates through the upper 30 m (100 ft) of soil profile, resulting in a design motion for liquefaction and embankment stability studies of 0.35g.

In the liquefaction assessment blowcounts from both the 1997 and the previous exploration programs were used to estimate the cyclic resistance ratio (CRR) for the soil on a test hole by test hole basis. Blowcounts from all SPTs were adjusted to an energy of 60 percent. An energy ratio of 80 percent was used for the automatic hammer; all other blowcounts were assumed to be measured at an energy of 60 percent. Other CRR correction factors, including those for overburden, fines correction, and earthquake magnitude were consistent with the latest recommendations of Robertson and Wride (1997).

The liquefaction potential, which is equivalent to the factor of safety against the occurrence of liquefaction, at each test hole location was determined by comparing the computed value of CRR to the cyclic stress ratio (CSR) caused by the design earthquake. If the liquefaction potential was 1.1 or lower, the soil was identified as having a high potential for liquefaction during a design earthquake. A check was then made to determine if the material with a high liquefaction potential met the grain size and plasticity criteria identified by Seed and Idriss (1982) as being necessary for a material to be liquefiable. Locations of high liquefaction potential were then plotted on the soil profile for the W-N Ramp to determine the trend in liquefaction.

Based on the blowcount analyses, it appears that liquefaction could develop between the groundwater location (i.e., elevation 18 m; 60 ft) and elevation 9 m (30 ft) at the W-N Ramp. This depth range encompasses all of Layer 2 and the upper portion of Layer 3. The potential for liquefaction is not, however, continuous within this elevation range. Rather, many of the blowcounts within the range suggest a low liquefaction potential, with the factors of safety against the occurrence of liquefaction in excess of 2. Individual points of liquefaction were then discounted if adjacent blowcounts were high, under the premise that re-distribution in porewater pressure would moderate the tendency for porewater pressure buildup. Likewise, blowcounts in areas where heave was specifically noted in the test hole log were also discounted.

From these interpretations, it was concluded that the soil between elevation 18 m (60 ft) and 15 m (49 ft) would be the most likely to liquefy on a relatively continuous basis; i.e., the entire layer would be liquefied at one time. Material between elevation 15 m (49 ft) and 9 m (30 ft) would undergo liquefaction on a more localized basis, with some zones of loose sands and silts liquefying but adjacent areas not liquefying.

Methods of Foundation Analyses

Foundation design studies were completed to determine the capacities of shallow and deep foundations that would likely be used during the widening project. The sizes for these foundations were provided by WSDOT's project manager. Approaches for the analyses were discussed with WSDOT prior to and during the analyses to confirm that the methods were generally consistent with WSDOT foundation design requirements.

Driven Pile Design

Axial pile capacities were determined for 460 and 610 mm (18 and 24 in) steel pipe piles. It was assumed that these piles would be driven with a closed end, and filled with concrete after driving. Analyses were conducted for these two pile sizes to determine the (1) axial capacity under static (service load) and seismic conditions, (2) the amount of settlement of a four-pile group under the service loads, and (3) soil parameters for lateral pile capacity determination.

Static Axial Capacity Determination

Both compressive and uplift capacities of the piles were determined. The unified method of design (Fellenius, 1996) was used to estimate compressive and uplift capacities. Coefficients for β and N_t used during these analyses are given in Table 6-2. No limitations were placed on the determination of side and end resistance when computing capacities. In some design methods a critical depth of 10 to 20 pile diameters is imposed, beyond which side friction and end resistance values do not increase (e.g., DM-7, 1982). However, for the depths involved and based on discussions by Fellenius and Altaee (1995), there seems to be considerable question whether the critical depth concept is appropriate.

Table 6-2. Summary of Coefficients for Driven Pile Design at W-N Ramp

Layer No.	Static Conditions		Seismic Conditions	
	β	N_t	β	N_t
1	0.35	-	0.35	-
2	0.30	-	0.15	-
3a (> elev. 9) 3b (< elev. 9)	0.45 0.45	55 55	0.15 0.45	- 55
4	0.32	-	0.32	-
5	0.30	35	0.30	35
6	0.45	60	0.45	60

In recognition that the soil layering seems to change between the southern piers (Piers 2 and 3) and the northern piers (Piers 4 and 5), separate analyses were completed to account for somewhat different soil layering and soil properties in each area.

The uplift capacity of the driven piles was assumed to be 80 percent of the friction along the side of the pile in compressive loading. This reduction is consistent with WSDOT 's standard practice.

Seismic Axial Capacity Determinations

Procedures used to estimate axial capacity under seismic loading differed from the method for estimating static capacity only in the assigned β value for Layer 2 and part of Layer 3. As discussed above, liquefaction is predicted at various depths in these layers under a design earthquake, the consequence of which will be reduction in the side and end resistance for the pile. It was assumed for the seismic axial capacity determination that liquefaction would occur between approximate elevations 18 and 9 (60 and 30 feet).

Throughout the liquefied zone, a reduced β value was used for side friction. The reduction in side resistance was introduced by using an undrained residual strength ratio (S_r/σ') equal to 0.15. This ratio was selected on the basis of information presented by Dobry and Baziar (1993) and in the draft proceedings from a 1997 National Science Foundation Workshop (NSF, 1997) dealing with the measurement of residual strengths in liquefied soil. A wide range of undrained strength ratios have been suggested for liquefied soil, and some individuals contend that the residual strength is not proportional to the effective overburden pressure. Considering the differences of opinion that currently exist, a check was also performed using the relationship between blowcount and residual strength suggested by Seed and Harder (1990). An undrained strength ratio of 0.15 results in undrained strengths that are not inconsistent with the range determined from the Seed and Harder relationship.

It was further decided that the toe of the pile should be located below the zone with a high risk of liquefaction (i.e., 18 to 9 m; 60 to 30 ft) to minimize the potential for excessive pile settlement during a design seismic event. No adjustments were made for potential buildup in porewater pressure below the liquefied zone. It was assumed that sufficient conservatism had been introduced by establishing the maximum toe elevation below the maximum predicted depth of liquefaction.

This approach to liquefaction was expected to be conservative. The actual effects of the assumption regarding side friction on compressive and uplift capacity are not significant, as the side resistance within this depth interval is relatively small, even under static conditions.

Settlement Estimates for Static Loading

Settlement estimates were made assuming that four piles would be required to support the pile cap for the column. The four-pile configuration was selected primarily on the basis of lateral stiffness, in the event that loss in soil strength occurs in Layer 2 and part of Layer 3 due to liquefaction as predicted. It was also assumed that the four piles would be spaced at 2 ½ to 3 diameters.

An equivalent footing approach was taken in estimating settlements. The size of the footing was defined by the perimeter of the pile group. Following discussions with WSDOT engineers, it was decided that the footing would be located at the neutral plane of a single pile, where the neutral plane was defined as the point at which the side friction for the pile equals the service load. A 2V:1H stress distribution was assumed below the footing.

Soil Parameters for Lateral Pile Loading

Procedures used to determine soil parameters for lateral-load analyses generally followed recommendations by Reese and others (e.g., Reese and Wang, 1989a). Modulus of subgrade reaction values were based on information presented in Lam and Martin (1986), which gives modulus of subgrade reaction values as a function of relative density for sands located above and below the water table. These parameters are appropriate for use in the computer programs LPILE and COM624.

For seismic loading the resistance of Layer 2 and Layer 3 was reduced to account for the likelihood of liquefaction under a design earthquake. While liquefaction could occur between elevation 18 and 9 m (60 and 30 ft), it appears that Layer 2, which makes up the upper 3 m (10 ft) of the liquefiable zone, is the most vulnerable. Within this layer a fully liquefied condition was assumed. The average corrected blowcount, $(N_1)_{60}$ for this layer was approximately 12, resulting in a β of 0.15 based on NSF (1997) or a strength of 12 kPa (250 psf) based on the lower bound of the relationship between residual strength and corrected SPT value given by Marcuson et al. (1990). Below approximate elevation 14 to 15 m (46 to 49 ft) the liquefied zone was assigned a friction angle midway between the liquefied and nonliquefied values. The basis for the reduced friction angle was that random locations of liquefaction were predicted in the lower portion of the layer. However, other locations within the same depth range did not liquefy. Realizing this, it was reasoned that some loss in lateral support capacity would potentially occur below elevations 14 to 15 m (46 to 49 ft), but more resistance would exist than a fully liquefied state.

Pile-group reduction factors were also defined to account for interaction between piles if the piles are closely spaced, as expected. The reduction factor will depend on the selected spacing ratio (i.e., ratio of center-to-center pile spacing to pile diameter). Significant differences in opinion currently exist within the profession regarding the form and amount of reduction to apply. Based on a recent survey of state departments of transportation (Brown at al., 1998), it was found that reduction factors given in references such as DM-7 (1982), the Canadian Foundation Engineering Manual (1985), and even the Federal Highways Administration (FHWA) Manual *Design and Construction of Driven Piles* (GRL, 1996) are generally viewed as resulting in too much reduction in stiffness. The p-multiplier procedure (e.g., Brown and Bollman, 1996) is currently thought to provide the most realistic representation of group effects, in the absence of dynamic analyses such as given in WSDOT's Design Manual for Foundation Stiffness Under Seismic Loadings (GeoSpectra, 1997).

Drilled Shaft Design

Axial capacities of three drilled shafts, with diameters of 1.22 m (4 ft), 1.83 m (6 ft), and 2.44 m (8 ft), were determined. It was assumed that a steel casing would be used during installation of these shafts, but that the casing would be removed as the concrete is placed. Analyses were conducted for each shaft diameter to determine (1) the axial capacity under static (service load) and seismic conditions, (2) the possible settlement of the shaft under service loads, and (3) soil parameters for lateral shaft capacity determination.

Static Axial Capacity Determination

The static capacity analyses for the shaft involved determination of side resistance, end bearing, and uplift resistance. Procedures suggested by the FHWA Manual *Drilled Shafts* (Reese and O'Neill, 1988) were generally followed when determining capacity. In this approach the end bearing of the shaft is determined from the product of the uncorrected blowcount (N) times a factor of 57.5 in kPa (or N X 0.6 in tsf), and the side friction for cohesionless soil is based on a computed β value.

Procedures used in the estimate of shaft side resistance deviated from recommendations given in the FHWA manual in one important area. When determining β values, the equation recommended in the FHWA manual was not followed. During a progress review meeting with WSDOT's geotechnical engineers, it was decided that the β values determined from the equation in the FHWA manual were too high in the upper layers of soil and possible too low in the lower layers. To obtain what were considered to be more representative β values for the soil conditions at the site and the likely construction methods, β was defined as the product of a lateral earth pressure coefficient (k) and the tangent of the interface friction angle.

Shaft capacities for the W-N Ramp were determined using the Ensoft computer program SHAFT1 (Reese and Wang, 1989b). This program computes shaft side and end resistance every 0.3 m (1 ft) throughout the depth of interest. Input to the program includes β and blowcounts for each layer. The values of β and the average N values used for the shaft capacities analyses are summarized in Table 6-3. No adjustments were made for shaft diameters greater than 1300 mm (50 in) based on discussions with WSDOT. As with the driven piles, uplift capacity was assumed to be 80 percent of the compressive capacity of the shaft.

Table 6-3. Summary of Coefficients for Drilled Shaft Design at W-N Ramp

Layer No.	Static Conditions				Seismic Conditions			
	Piers 2 & 3		Piers 4 & 5		Piers 2 & 3		Piers 4 & 5	
	β	N	β	N	β	N	β	N
1	0.32	10	0.32	10	0.32	-	0.32	-
2	0.27	8	0.27	8	0.15	-	0.15	-
3a (> elev. 9) 3b (< elev. 9)	0.42 0.42	27 32	0.36 0.36	22 22	0.15 0.42	- 22	0.15 0.36	- 22
4	0.29	8	0.29	8	0.29	8	0.29	8
5a (>elev. -8) 5b (<elev. -8)	0.29 0.29	7 15	0.29 0.29	7 15	0.29 0.29	7 15	0.29 0.29	7 15
6	0.54	75	0.54	75	0.54	75	0.54	75

Seismic Axial Capacity Determinations

Procedures used to estimate the axial capacity of the shaft under seismic loading differed from the method for estimating static capacity only in the assigned β value for Layer 2 and part of Layer 3. As discussed previously for driven piles, liquefaction is predicted at various depths in these layers under a design earthquake, the consequence of which is reduction is the strength of the layer. It was assumed that the β value would be reduced to 0.15 between elevations 18 and 9 m (60 and 30 ft). The rationale for the selection of β of 0.15 is the same as that given for driven piles. Also similar to the driven pile, it was concluded that the toe of the shaft should be located below the maximum predicted depth of liquefaction.

Settlement Estimates for Static Loading

Settlement estimates were made assuming that a single shaft would support each column. An equivalent footing approach was taken in estimating settlements. The size of the footing was defined by the perimeter of the shaft. This footing was located at the neutral plane of the shaft. As noted before, the neutral plane was defined as the point at which the side friction for the shaft equals the service load. A 2V:1H stress distribution was assumed below the equivalent footing.

Soil Parameter for Lateral Pile Loading

Procedures used to determine soil parameters for lateral-load analyses were the same as those used for driven piles. After discussions with WSDOT's geotechnical engineers, it was decided that no adjustment factors would be given to account for the potential effects of shaft diameters greater than 0.6 m (2 ft), as has recently been suggested in some studies (e.g., ATC, 1996). These parameter are appropriate for use in the computer programs LPILE and COM624.

As with the driven piles, the strength of the soil between elevation 18 and 9 m (60 and 30 ft) was reduced to account for the likelihood of liquefaction under a design earthquake. While liquefaction could occur throughout the elevation range, the upper 3 m (10 ft) were considered most vulnerable. Within this layer a fully liquefied condition, with a residual strength of 12 kPa (250 psf), was assumed. The lower portion of the range was assigned a friction angle midway between the liquefied and nonliquefied values. The basis for this was the same as discussed previously for driven piles.

Abutment Design

To facilitate the widening, it will be necessary to increase the width of the embankment side slopes by approximately 4 m (13 ft). Abutment footings will also have to be constructed in the approach fill to support the new bridge width. In the case of the abutment fill, analyses were performed to determine the stability of the new side slopes and end slopes under static and seismic loading. For the abutment footings, it was necessary to determine allowable bearing pressures and strain-compatible dynamic soil properties for the footing. Procedures used to evaluate these requirements are summarized below.

Abutment Stability

The stability of the side slopes and end slopes for the abutment fill under static and seismic loading was determined by conducting stability analyses using the computer program PCSTABL (Siegel, 1974). For these analyses the groundwater was assumed to be located at elevation 18 (60 ft), which is roughly 3 m (10 ft) below the existing ground surface. The slope of the embankment was assumed to be 2H:1V, which was similar to the end slope and somewhat steeper than the side slopes. Properties of the embankment material and underlying soils were as defined previously within the discussion of Engineering Soil Properties.

For the seismic case pseudo static analyses were conducted using PCSTABL. In this approach the seismic coefficient was varied until a factor of safety approximately equal to 1.0 was defined. Properties were similar to those used for the static analyses, except that Layer 2 was assigned a residual strength equal to 0.15 times the effective overburden pressure (i.e., $S_r = 0.15\sigma'$). The basis for the residual strength determination was presented previously in the discussion for Driven Pile Design. A 3-m (10 ft) layer was used to constrain the depth of the failure surface to the zone where continuous liquefaction was expected.

Estimates of deformation during the seismic event were made using the Newmark simplified method. With this method, an approximate estimate of deformation can be obtained from published relationships between the predicted deformation and the ratio of yield acceleration to peak acceleration.

Allowable Footing Pressures and Dynamic Properties

Each end of the existing bridge is supported on an abutment wall that is supported on a 1.5-m (5 ft) wide strip footing extending across the complete width of the bridge. This footing is located approximately 3 m (10 ft) below the roadway surface. It is anticipated that a similar size footing at the same depth will be used for the widening. Allowable bearing pressures for this footing were determined using conventional bearing capacity theory with allowances for the sloping face of the end abutment. It is understood that the lateral earth pressures for the abutment wall will be based on WSDOT's standard wall design.

Shear modulus, material damping, and Poisson's ratio values were estimated based on recommendations given in the FHWA Manual *Seismic Design of Bridge Foundations* (Lam and Martin, 1986). For these analyses the low-strain shear modulus was selected on the basis of average blowcounts recorded during the SPTs within one footing width below the planned footing elevation. An average shearing strain of 0.02 to 0.2 percent was used to adjust for the level of shearing strain expected during a design event.

Recommendations

This presentation of recommendations is separated into two sections. The first covers the foundation systems, and the second involves construction considerations. While the discussion of construction is limited, recommendations given for design of the foundation systems are dependent on the methods used and observations made during construction. For this reason it is critical that any changes in either site conditions encountered during construction or procedures used during construction be brought to the attention of CH2M HILL in order that the following foundation recommendations can be confirmed for the observed conditions or methods.

Foundations

The methods of analyses described in the preceding section were used to develop geotechnical recommendations for design of driven pile and drilled shaft foundations, abutment footings, and abutment slopes under static and seismic loading conditions. These recommendations are based on best estimates of soil properties. Appropriate consideration should be given to the possibility of different soil properties and soil behavior during selection of factors of safety.

Driven Piles and Drilled Shafts -- Static Loading

The interior columns for the bridge can be supported using either driven piles or drilled shafts.

Axial Capacity: Figures 6-3 through 6-12 present **ultimate** axial capacity versus depth plots for each pile and shaft size. It is emphasize that these capacities are ultimate values; they have not been reduced with factors of safety. The maximum ultimate capacity for driven piles is limited to 4,500 kN (500 tons) to keep ultimate capacity within the range of applicability of the dynamic formula in Section 6-05 of WSDOT's Standard Specifications.

Allowable values can be determined by applying a factor of safety to the capacities given in Figures 6-3 to 6-12. Table 6-4 provides recommended factors of safety for design. As shown in this table, the factor of safety should be selected on the basis of the type of field monitoring that is done before or during pile or shaft installation. It is understood that

WSDOT normally will monitor pile drivability or shaft construction; however, if test piles are driven or a static load test were performed, lower factors of safety would be appropriate.

Table 6-4. Recommended Factors of Safety at W-N Ramp

Field Confirmation	Driven Piles		Drilled Shafts	
	Compressive Loading	Uplift Loading	Compressive Loading	Uplift Loading
None	3	3	4	4
Standard WSDOT	2.5	1.5	2.5	1.5
Test Piles/PDA	2.25	1.4	-	-
Static Load Test	2.0	1.3	2.0	1.3

Minimum and maximum pile or shaft toe elevations should be used with Figures 6-3 through 6-12 to assure development of the required capacities and to limit settlements. Table 6-5 provides a summary of the minimum and maximum toe elevations for the bearing layers. These elevations were established (1) to avoid locating the toe of the driven pile or drilled shaft in what was thought to be a more compressible material (e.g., Layers 2 and 4), (2) to locate the toe of the shaft or driven pile below the maximum anticipated depth of liquefaction, and (3) in the case of drilled shafts to limit construction depths to lengths that WSDOT believes can be achieved.

Layers that should not be used for end bearing due to soil type or liquefaction potential are identified with "NA", meaning not appropriate. It is important to note that the drilled shafts at Piers 4 and 5 should not be located above elevation 0. This elevation requirement is imposed because of the uncertain consistency of Layer 3 at Piers 4 and 5. Blowcounts recorded during the 1997 field exploration program were often low within this depth zone. Although it is possible that the low blowcounts were due primarily to heave during the drilling program, the possibility of very loose materials could not be ruled out. After discussing this issue with WSDOT's geotechnical engineers, it was decided that the toe of the shafts should be located below the zone where low blowcounts were recorded. Should this requirement have significant cost implications, then it may be necessary to conduct further explorations at Piers 4 and 5 to reconcile this issue. If additional explorations are conducted, it would be preferable to conduct these explorations with a cone penetrometer to obtain a continuous determination of soil resistance with depth.

For any layer, a four-pile group or drilled shaft founded between the minimum and maximum toe elevations is expected to develop the capacities given in Figures 6-3 through 6-12 with settlements under service loading of less than 25 mm (1 in).

Table 6-5. Summary of Minimum and Maximum Toe Elevations at W-N Ramp

Layer Number	Driven Piles				Drilled Shafts			
	Minimum Elevation (m)		Maximum Elevation (m)		Minimum Elevation (m)		Maximum Elevation (m)	
	Piers 2 & 3	Piers 4 & 5	Piers 2 & 3	Piers 4 & 5	Piers 2 & 3	Piers 4 & 5	Piers 2 & 3	Piers 4 & %
1	NA	NA	NA	NA	NA	NA	NA	NA
2	NA	NA	NA	NA	NA	NA	NA	NA
3	9	9	4	6	9	NA	5	NA
4	NA	NA	NA	NA	NA	NA	NA	NA
5	-3	-3	NR*	NR	-3	0	-10	-10

* Not Restricted

Lateral Capacity: Soil properties that should be used for non-seismic lateral pile capacity analyses are summarized in the LPILE/COM624 forms given in Tables 6-6 and 6-7. The elevation of the top of the first layer should be the bottom of the pile cap for driven piles or 1.5 m (5 ft) below the ground surface at the shaft location.

Table 6-6. LPile/COM624 Parameters for Service Loading at W-N Ramp - Piers 2 & 3

Layer No.	Type of Soil	Layer Elevation				Effective Unit Weight		Cohesion		Friction Angle	Coefficient of Subgrade Reaction		Soil Type
		Upper		Lower									
		(m)	(ft.)	(m)	(ft.)	(kN/m³)	(pcf)	(kPa)	(psf)	(degr.)	(MN/m³)	(pci)	
1	Sand	-	-	17	56	19.6	125	0	0	33	24	90	4
2	Silt	17	56	14	46	8.3	53	0	0	29	3	10	4
3	Sand w/ gravel	14	46	1	3	9.8	63	0	0	33	16	60	4
4	Silty Sand	1	3	-2	-7	9.1	58	0	0	30	9	35	4
5	Sand w/ silt & gravel	-2	-7	-13	-43	9.1	58	0	0	30	9	35	4
6	Sandy Gravel	-13	-43	-	-	10.5	67	0	0	35	23	85	4

Table 6-7 LPile/COM624 Parameters for Service Loading at W-N Ramp - Piers 4 & 5

Layer No.	Type of Soil	Layer Elevation				Effective Unit Weight		Cohesion		Friction Angle	Coefficient of Subgrade Reaction		Soil Type
		Upper		Lower									
		(m)	(ft.)	(m)	(ft.)	(kN/m³)	(pcf)	(kPa)	(psf)	(degr.)	(MN/m³)	(pci)	
1	Sand	-	-	18	56	19.6	125	0	0	33	24	90	4
2	Silt	18	56	15	49	8.3	53	0	0	29	3	10	4
3	Sand w/ gravel	15	49	5	17	9.8	63	0	0	30	9	35	4
4	Silty Sand	5	17	0	0	9.1	58	0	0	30	9	35	4
5	Sand w/ silt & gravel	0	0	-14	-49	9.1	58	0	0	30	9	35	4
6	Sandy Gravel	-14	-49	-	-	10.5	67	0	0	35	23	85	4

Group reduction factors should be applied if driven piles have spacing ratios of less than five diameters. The group reduction factors given in the following table were developed from Brown and Bollmann (1996). These values apply to the average stiffness of the pile group.

Table 6-8. Group Efficiency Factors for Driven Piles at N-W Ramp

Row Spacing	3-Pile Group	4-Pile Group	6-Pile Group
3 diameters	0.75	0.65	0.60
4 diameters	0.90	0.85	0.80
5 diameters	1.0	1.0	0.95

Driven Piles and Drilled Shafts -- Seismic Loading

Figures 6-13 through 6-22 present capacity versus depth plots for each pile and shaft size for seismic loading. These plots can be used with seismic loads to confirm that adequate axial capacity still exists when liquefaction occurs in the upper soil layers. In view of the conservative approach used in considering liquefaction for the axial capacity determinations, a factor of safety of 1.0 and 1.3 should be adequate for driven piles and drilled shafts, respectively, during a seismic event. Realizing the high liquefaction potential in Layers 2 and the upper portions of Layer 3, a minimum toe elevation is established at elevation 9 m (40 ft).

The pile or shaft foundation system could settle during the seismic event. This settlement is expected to result from two sources: (1) the added pile or shaft loads resulting from the inertial response of the structure and (2) densification of the upper portions of Layer 5.

Settlement from added bridge loads is expected to be small. Settlement from the densification of loose materials in the upper portion of Layer 5 could result in up to 50 mm (2 in) of settlement within Layer 5. Driven piles or drilled shafts founded above Layer 5 could settlement this amount. Similar amounts of settlement would also be expected to occur at the approach fills. If the driven piles or drilled shafts are founded in Layer 6, then settlement of the interior piers could occur due to drag loads as loose soils densify; however, this settlement is expected to be small. Settlement would still occur at the approach fills, resulting in differential movements between Pier 1 and Pier 2 and between Pier 3 and Pier 4. The amount of this differential movement could be as much as 50 mm (2

Soil properties that should be used for lateral pile capacity analyses during seismic loading are summarized in Tables 6-9 and 6-10. Group adjustment factors discussed above for static loading should be applied. Inasmuch as the phasing between liquefaction and the maximum inertial forces on the bridge structure is difficult to predict, it is recommended that seismic analyses include lateral capacity evaluations for two cases: (1) a nonliquefied case, which is equivalent to the static case (Tables 6-6 and 6-7), and (2) the seismic case. Design should be based on the more critical of the two.

Table 6-9. LPile/COM624 Parameters for Seismic Loading at N-W Ramp - Piers 2 & 3

Layer No.	Type of Soil	Layer Elevation				Effective Unit Weight		Cohesion		Friction Angle	Coefficient of Subgrade Reaction		Soil Type
		Upper		Lower									
		(m)	(ft.)	(m)	(ft.)	(kN/m³)	(pcf)	(kPa)	(psf)	(degr.)	(MN/m³)	(pci)	
1	Sand	-	-	17	56	19.6	125	0	0	33	24	90	4
2	Silt	17	56	14	46	8.3	53	12*	250	-	-	-	1
3a	Sand w/ gravel	14	46	9	30	9.8	63	0	0	21	5	18	4
3b	Sand w/ gravel	9	30	1	3	9.8	63	0	0	33	16	60	4
4	Silty Sand	1	3	-2	-7	9.1	58	0	0	30	9	35	4
5	Sand w/ silt & gravel	-2	-7	-13	-43	9.1	58	0	0	30	9	35	4
6	Sandy Gravel	-13	-43	-	-	10.5	67	0	0	35	23	85	4

Note: For Layer 2, assume $\varepsilon_{50} = 0.02$ mm/mm

Table 6-10. LPile/COM624 Parameters for Seismic Loading at N-W Ramp - Piers 4 & 5

Layer No.	Type of Soil	Depth to Boundary				Effective Unit Weight		Cohesion		Friction Angle	Coefficient of Subgrade Reaction		Soil Type
		Upper		Lower									
		(m)	(ft.)	(m)	(ft.)	(kN/m³)	(pcf)	(kPa)	(psf)	(degr.)	(MN/m³)	(pci)	
1	Sand	-	-	18	60	19.6	125	0	0	33	24	90	4
2	Silt	18	60	15	49	8.3	53	12*	250	-	-	-	1
3a	Sand w/ gravel	15	49	930	62	9.8	63	0	0	21	5	18	4
3b	Sand w/ gravel	9	30	5	17	9.8	63	0	0	30	9	35	4
4	Silty Sand	5	17	0	0	9.1	58	0	0	30	9	35	4
5	Sand w/ silt & gravel	0	0	-14	-49	9.1	58	0	0	30	9	35	4
6	Sandy Gravel	-14	-49	-	-	10.5	67	0	0	35	23	85	4

* Note: For Layer 2, assume ε_{50} = 0.02 mm/mm

Abutment Footings

The abutment footing should be designed for an allowable bearing pressure of 290 kPa (3 tsf). With this loading the settlements are expected to be less than 25 mm (1 in). Roughly half of the settlements is expected to occur during construction of the footing and abutment wall. For seismic loading (i.e., Load Case 7) the allowable pressure on the abutment footing can be increased by a factor of 2.

Shear modulus, material damping, and Poisson's ratio values given in Table 6-11 are recommended for determining stiffness values for seismic design. These values were developed using a shear wave velocity of 250 mps (820 fps), which results in a low-strain shear modulus of approximately 120 MPa (2,500 ksf).

Table 6-11. Dynamic Soil Properties for Abutment Footing at W-N Ramp

Mode of Vibration	Shearing Strain = 0.02%	Shearing Strain = 0.2%
Shear Modulus	80 MPa (1,700 ksf)	30 MPa (630 ksf)
Material Damping	5%	12%
Poisson's Ratio	0.35	0.35

In the event that future design studies determine that strip footings cannot be used, because of the available room or for whatever other reason, it would be possible to use drilled shafts or driven piles to support the abutment wall. Axial and lateral capacity information presented in this chapter for the closest pier can be used for drilled shaft and driven pile designs at the abutment should a spread footing not be feasible.

Embankment Slopes

The side slopes in the widened area should not exceed 2.5H:1V, which is the maximum existing side slope. End slopes should not exceed 2H:1V, which is also the existing slope steepness. For these slope angles the factor of safety for static loading will be greater than 1.5.

During a design seismic event, deformations of the end slopes and side slopes could occur. The amount of deformation is estimated to be less than 0.3 m (1 foot). Deformations at the end slopes could impose loads on the foundations for the columns. These loads would be imposed on the existing foundations, as well as the foundations for the widening project. In the event that at some future date a seismic retrofit is performed for the widened bridge, the retrofit should consider the potential effects of these additional loads on the foundation system. These effects could be evaluated by conducting lateral analyses of pile or shaft foundations with an imposed load from the moving soil. If the level of deformations cannot be tolerated, various ground improvement methods could be considered as part of the overall retrofit program.

Construction

Construction of the foundations for the widening project requires consideration of a number of issues related to both quality control and difficulties associated with construction. A number of these issues specific to this project site are summarized below. In most cases the contractor should be made aware of these issues or requirements at the time of bidding.

Driven Piles

The primary issues and requirements associated with the use of driven piles are as follows:

- The potential for wood and cobbles exists throughout the soil profile, and particularly in Layers 3 to 6. While these conditions were not widespread, sufficient cases were noted during the drilling of test holes to warrant consideration during the contracting of pile installation. Pile driving contractors should be advised of this possibility within the special provisions.
- In recognition of the uncertainties of axial pile capacity between the southern and northern piers, test piles should be installed prior to establishing pile order lengths. These test piles should be of the same size and should be driven with the same equipment as will be used during construction.

Table 6-10. Recommended Test Pile Program at W-N Ramp

Bridge	Pier Number	Number of Tests
E Ramp	2	1
	3	1
	4	1
	5	1

- Groundwater could be located within 1.5 to 3 m (5 to 10 ft) of the ground surface. Depending on the location of the bottom of the pile cap, excavations below the ground water elevation could be required. The permeability of Layer 1, in which the pile cap would likely be located, is expected to be high. With this high permeability, it would be essential for the contractor to have identified procedures for handling excess water in the excavation. If winter construction is anticipated, seals may be required to control water. If summer construction occurs, dewatering systems may be sufficient to control water.

- Site access will be very restricted for this bridge. It will likely require lane closures and, possibly, rerouting of traffic.

Drilled Shafts

The primary construction issues and requirements for drilled shaft will be as follows:

- The water table is very high for the site and soils are primarily cohesionless. This will necessitate the use of steel casing from the ground surface to the maximum depth of construction. It is critical that the casing be removed during placement of concrete, as friction values used for shaft capacity design are based on a soil-concrete interface and not a soil-steel interface. If the casing cannot be removed, shaft side resistance could decrease by as much as 50 percent.

- Shaft lengths could be up to 30 m (100 ft) in length to meet lateral fixity requirements during seismic events. For these lengths quality control during placement of concrete will be critical. Realizing the potential consequences of poor quality control, WSDOT should plan to conduct sonic crosshole logging in each shaft following construction.

- Access will be a significant construction consideration for each pier location.

Abutment Footing

The primary issues related to the construction of the abutment footing are as follows:

- It will likely be necessary to use sheet piling to support the existing abutment fill during excavation for and construction of the new footing. The depth of excavation for the footing will be 3 to 4 m (10 to 13 ft), if the footing is similar in size to the existing footing (i.e., 1.5 m; 5 ft). However, if a wider footing is needed to needed to meet slope-setback requirements, deeper excavations may be required.

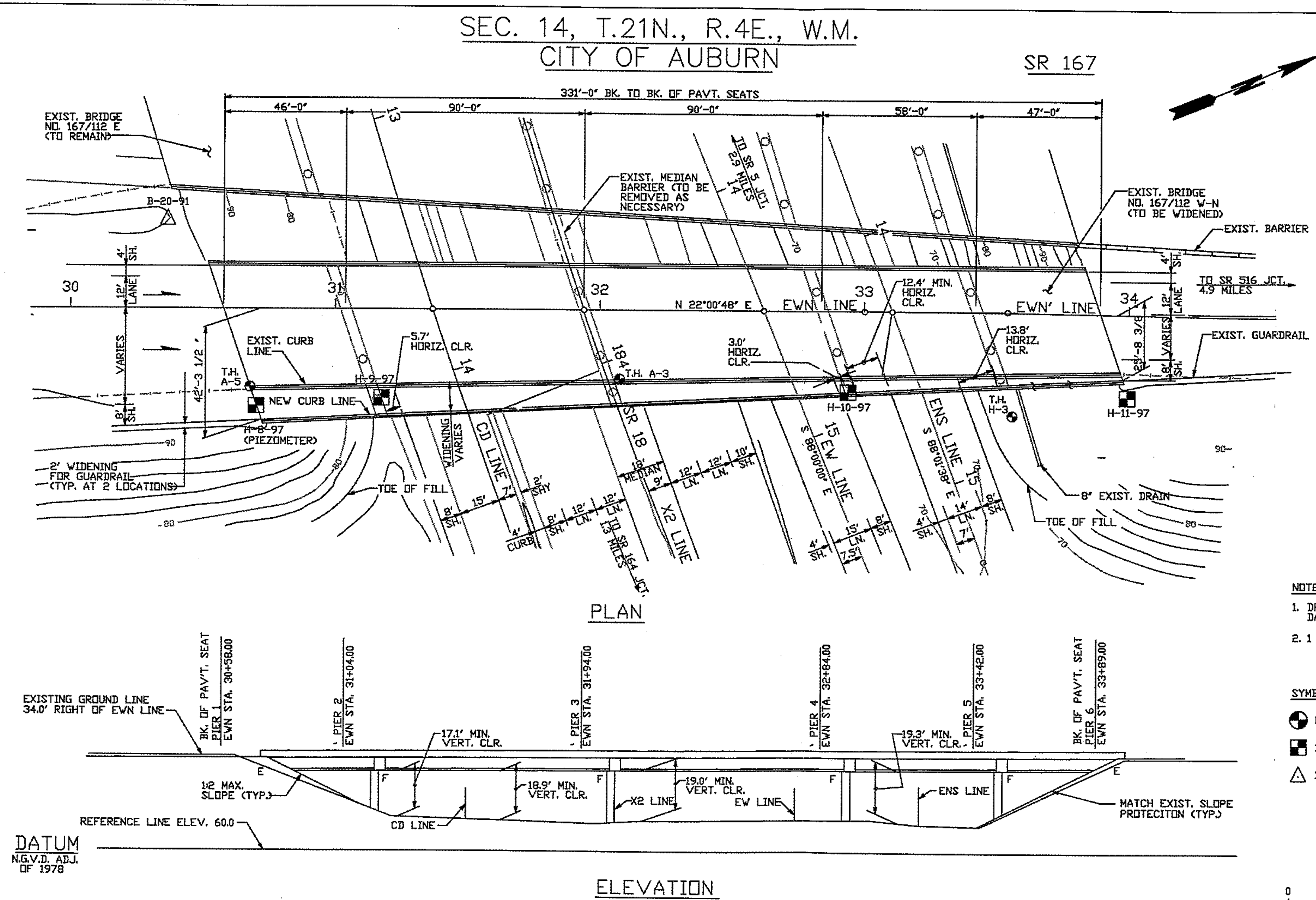
- In the event that the new footing is located below the existing footing, special care will be required to avoid loss of footing support for the existing footing during construction. Sheet piling or other support methods are available to provide this support. However, it should be made clear in the special provisions that support of the existing footing must be maintained. It would be desirable to survey the vertical elevations of the abutment wall before construction to be able to quantify any movement that does occur.

- Considering the potential for layers of siltier materials at the base of the planned footing excavation, the footing excavation should be carried to at least 0.3 m (1 ft) below the planned base of the footing. Crushed ballast should be compacted to the base of the footing to assure good drainage and high base friction.

Abutment Slopes

The primary construction issues and requirements related to the abutment slopes are as follows:

- The new side slope fill should be keyed into the existing fill by cutting benches into the existing embankment, as specified in WSDOT's standard specifications.
- Concrete slope protection matching the existing slope protection should be used to prevent ravelling of embankment materials beneath the bridge.



NOTES

1. DRAWING ADAPTED FROM PRELIMINARY BRIDGE PLAN DATED OCTOBER 1997. FINAL PLANS MAY VARY.
2. 1 FOOT = 0.305 METERS

SYMBOLS

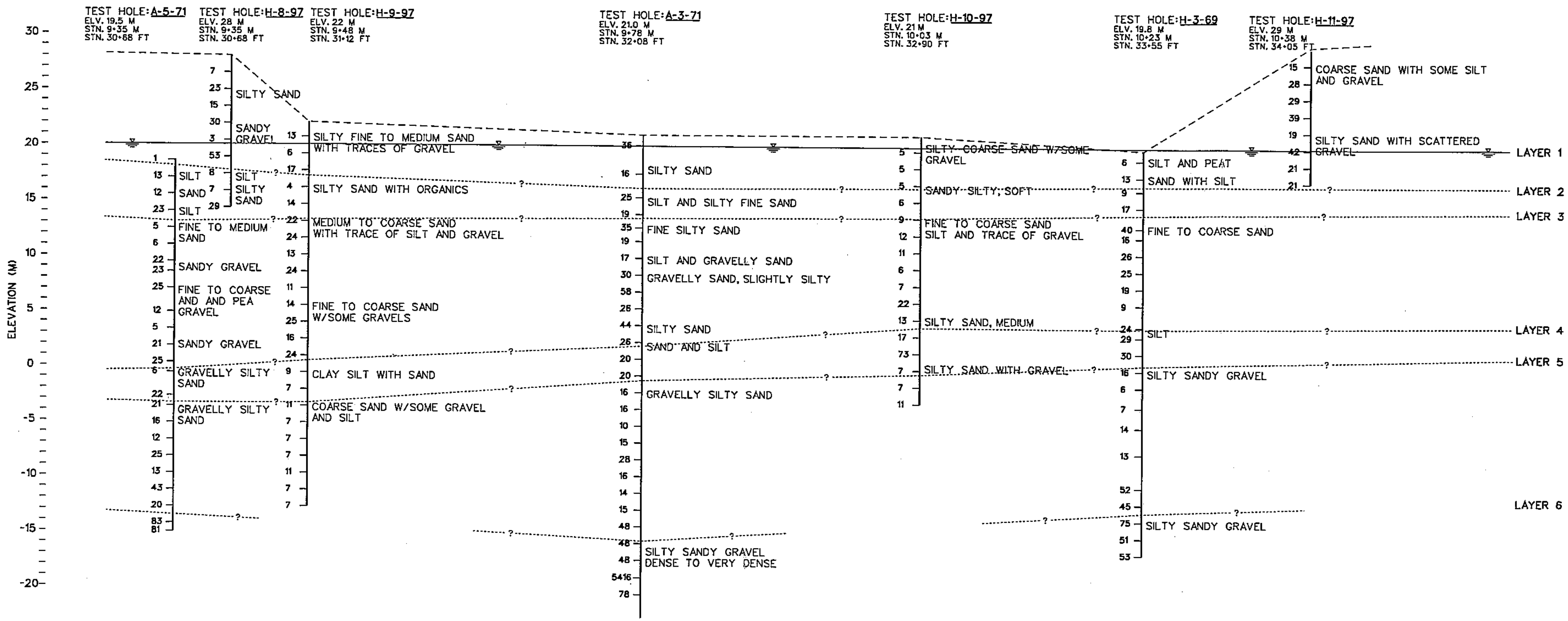
- EXISTING WSDOT TEST HOLE
- 1997 WSDOT TEST HOLE
- 1991 TERRA TEST HOLE

Washington State Department of Transportation

CH2MHILL

FIGURE 6-1
W-N RAMP WIDENING
TEST HOLE LOCATIONS

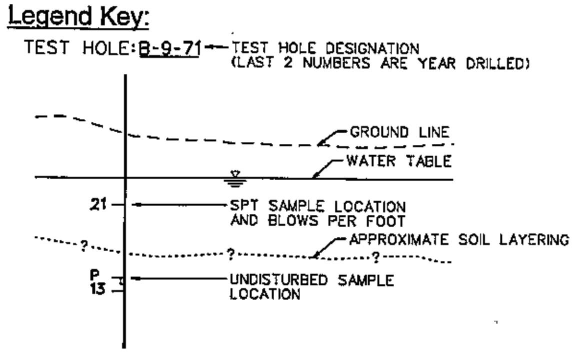
C.S. 1765, PROJECT NO. OL-2305, NORTHWEST REGION, 15TH ST. S.W. TO 15TH ST. NW, HOV LANE – STAGE 3, SR 167, BR. NO. 167/112 W-N RAMP WIDEN.



Notes:

1. Soil layering is based on interpretations from soil test hole logs and engineering judgment. Actual conditions within and between test holes could differ from those indicated.
2. Water table elevation based on maximum estimated conditions. Actual elevation could be as much as 3 meters below identified elevation.

Legend Key:



0 5 10 15
SCALE: 1"=10M

Figure 6-2

Soil Profile For
Bridge No. 167/112 W-N Ramp
Geotechnical Report
SR-167, OL-2305
15th Avenue SW To 15th Avenue NW
HOV Widening Project

CH2MHILL

116184.G4 FILE NAME: 11618402.dlv 10-FEB-1998 07:36:32

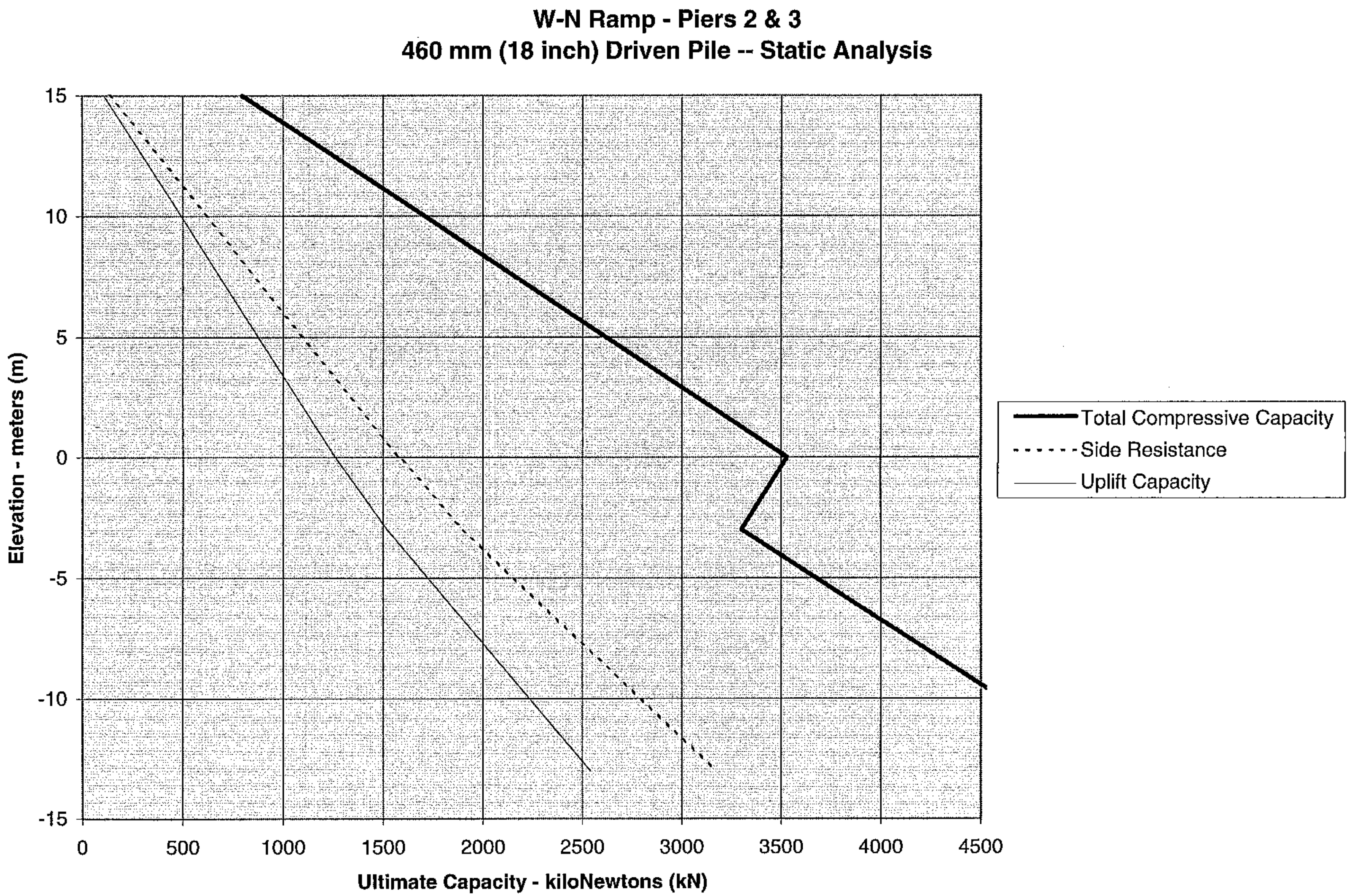


Figure 6-3. Ultimate Pile Capacity for 460 mm (18 inch) Driven, Closed-End Steel Piles for Piers 2 & 3 at W-N Ramp – Static Analysis

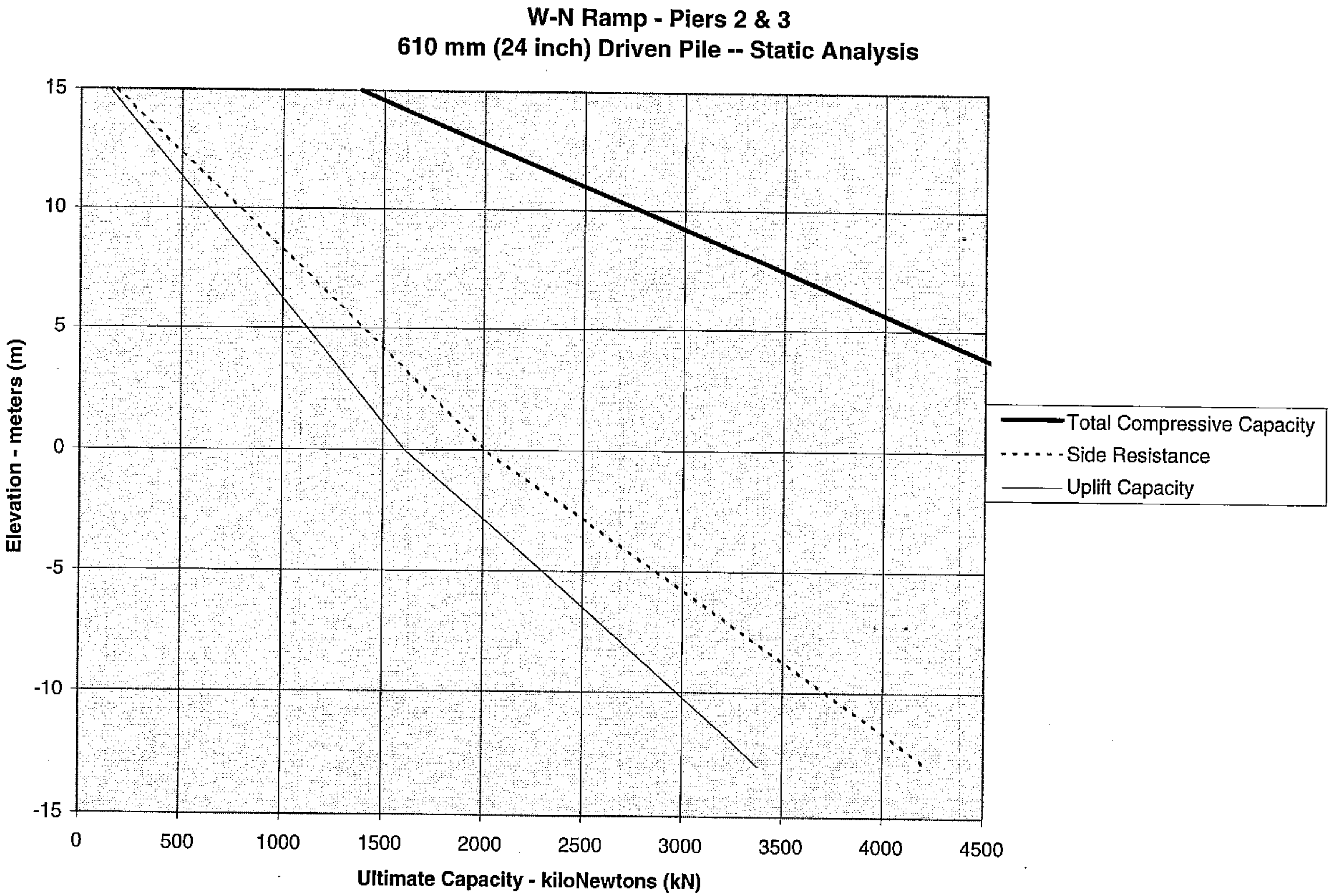


Figure 6-4. Ultimate Pile Capacity for 610 mm (24 inch) Driven, Closed-End Steel Piles for Piers 2 & 3 at W-N Ramp – Static Analysis

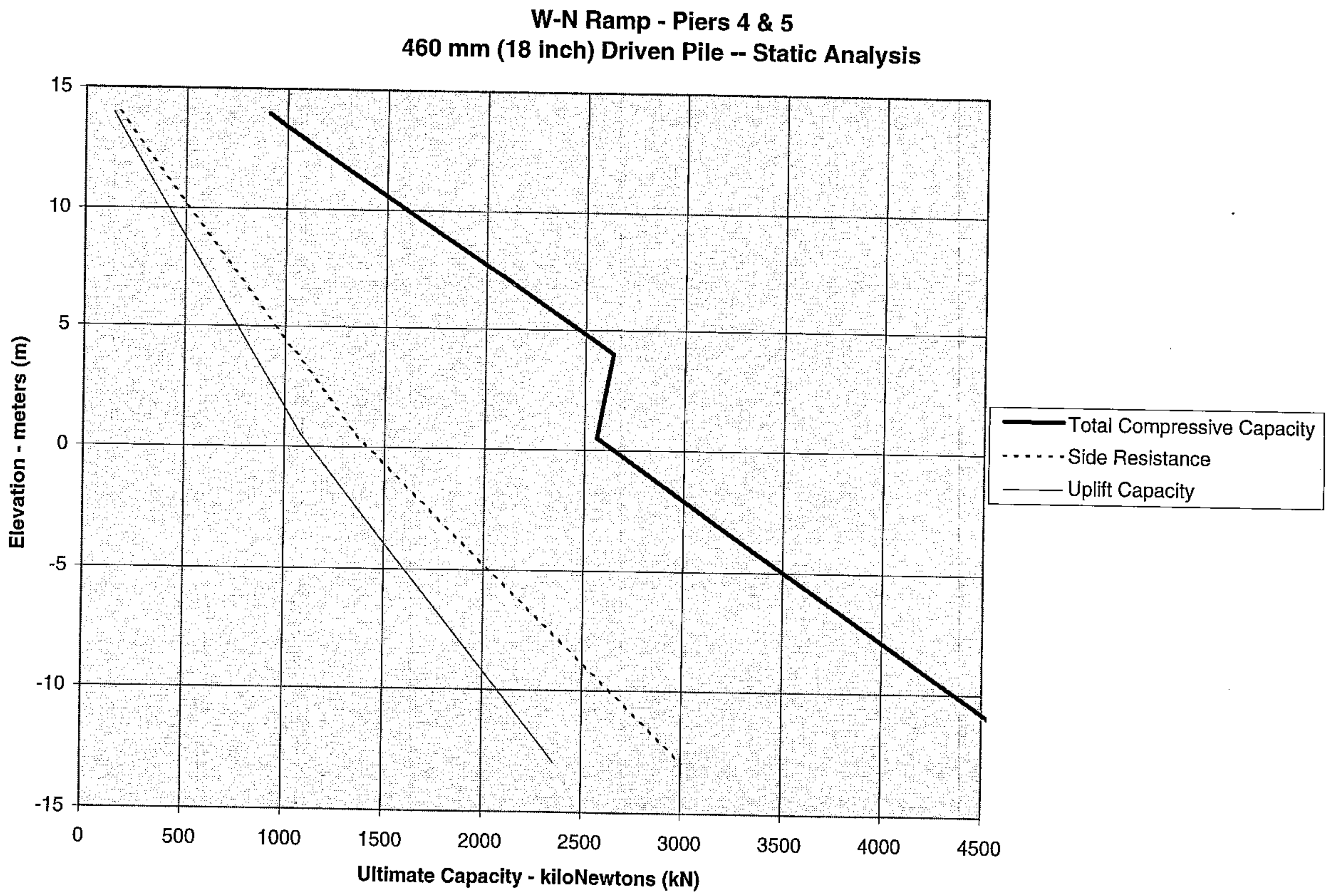


Figure 6-5. Ultimate Pile Capacity for 460 mm (18 inch) Driven, Closed-End Steel Piles for Piers 4 & 5 at W-N Ramp – Static Analysis

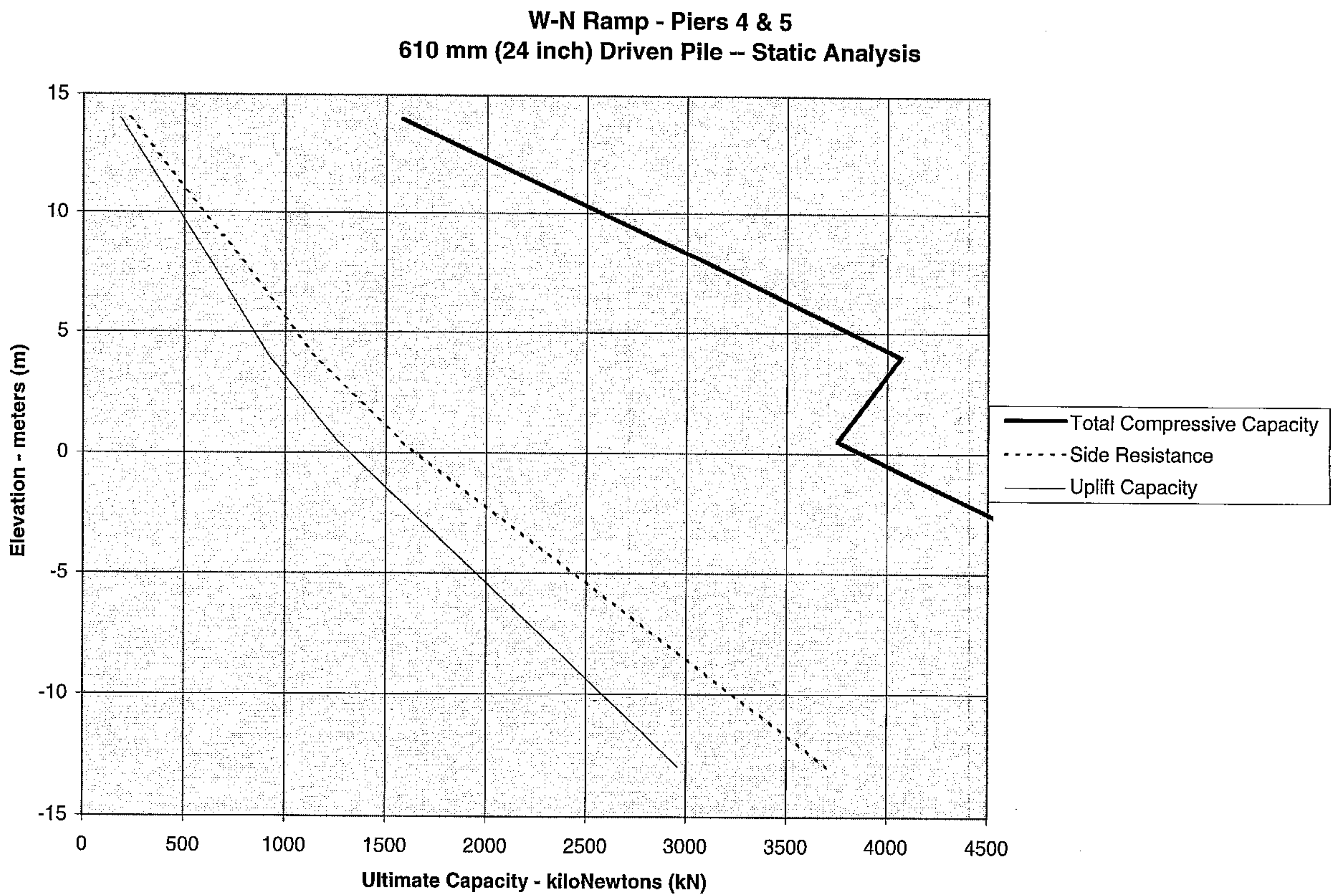


Figure 6-6. Ultimate Pile Capacity for 610 mm (24 inch) Driven, Closed-End Steel Piles for Piers 4 & 5 at W-N Ramp – Static Analysis

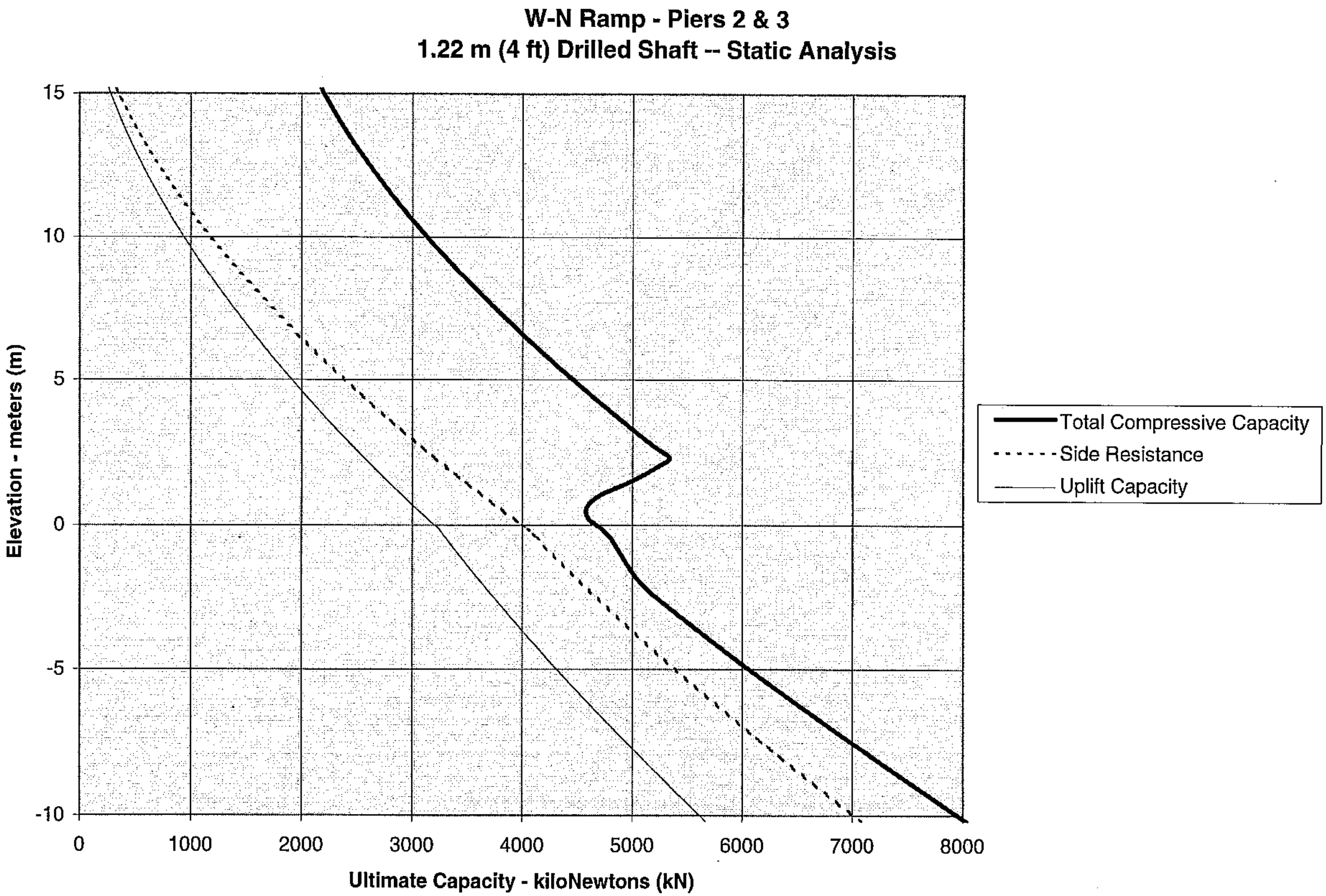


Figure 6-7. Ultimate Capacity of 1.22 m (4 ft) Drilled Shaft for Piers 2 & 3 at W-N Ramp – Static Analysis.

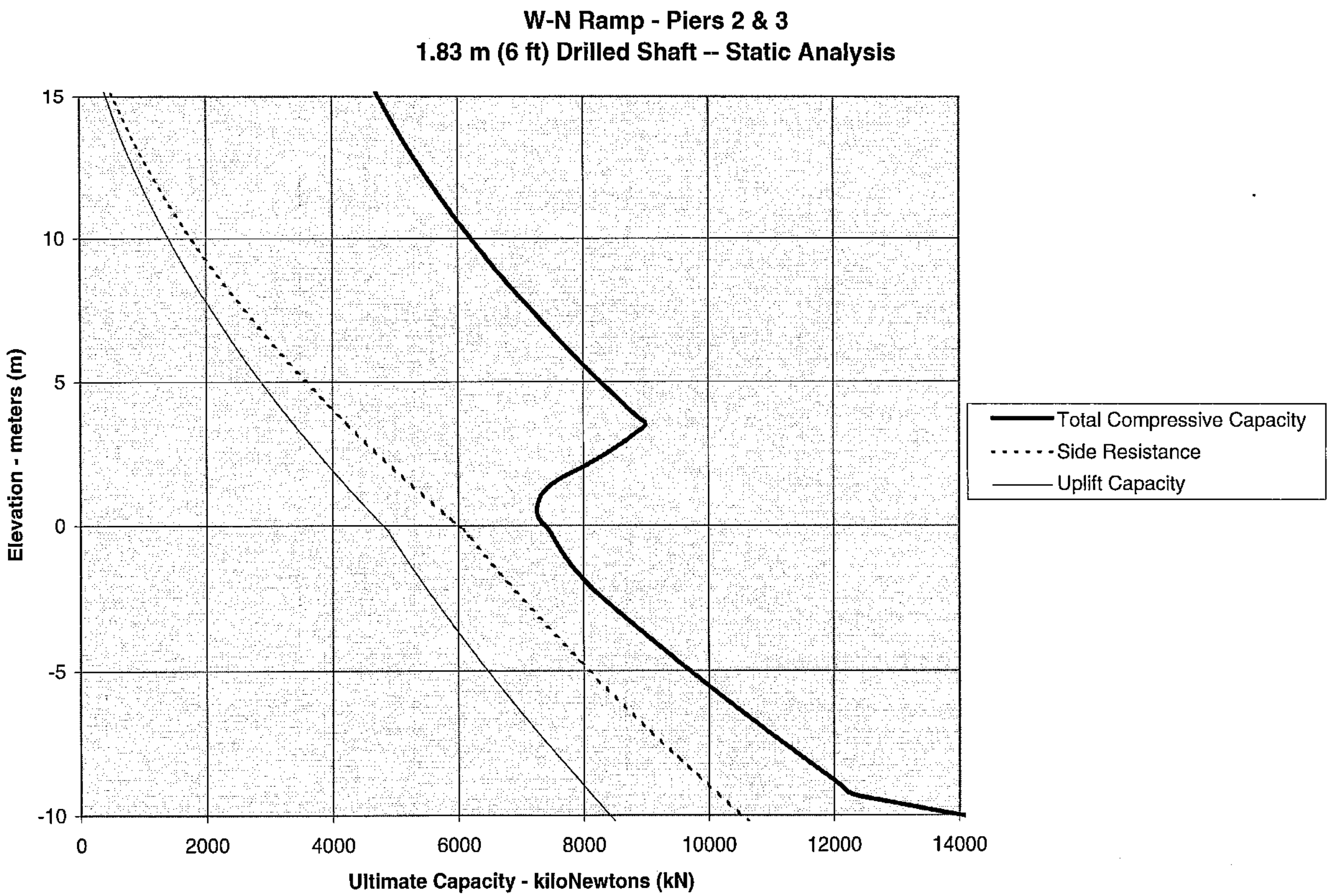


Figure 6-8. Ultimate Capacity of 1.83 m (6 ft) Drilled Shaft for Piers 2 & 3 at W-N Ramp – Static Analysis

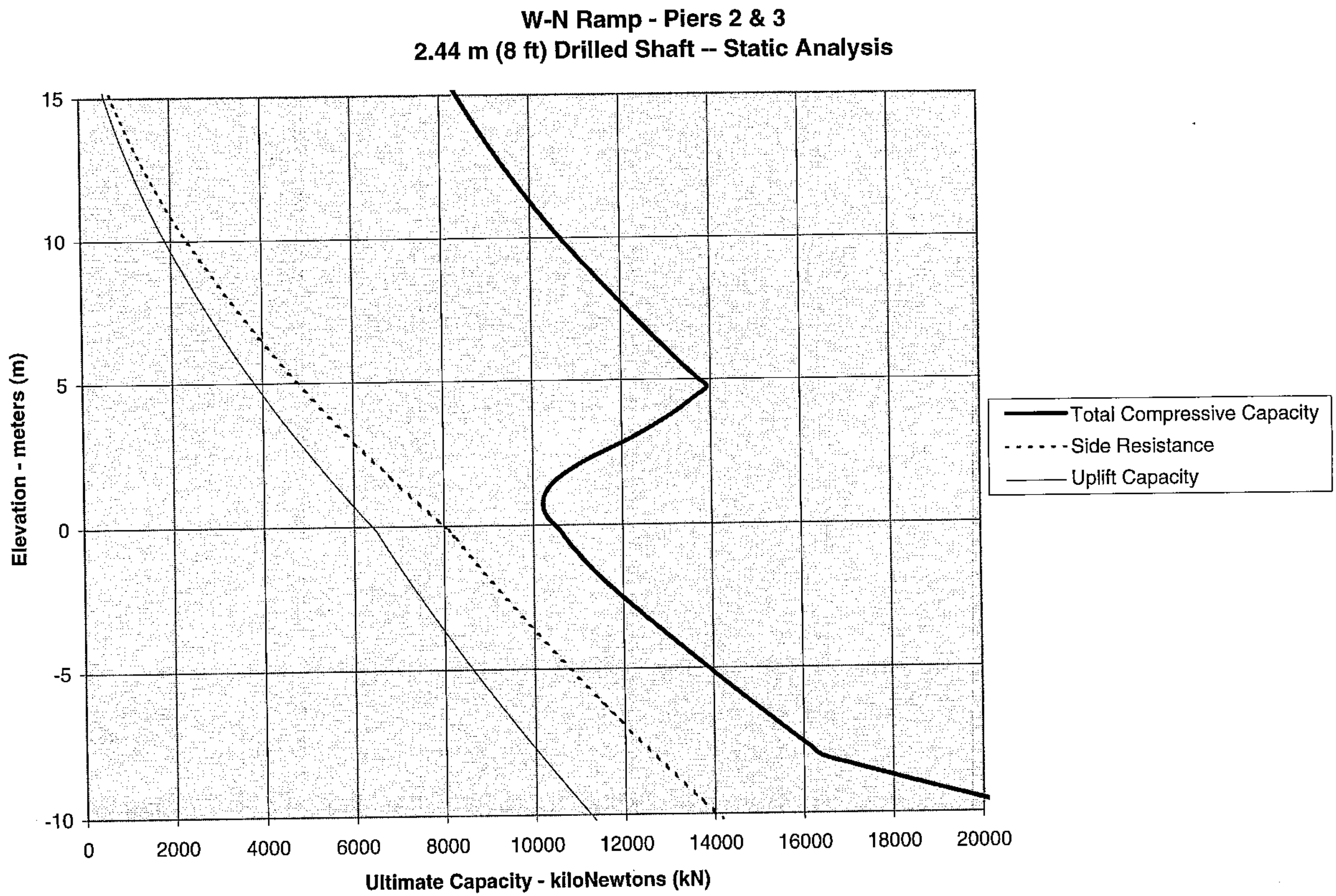


Figure 6-9. Ultimate Capacity of 2.44 m (8 ft) Drilled Shaft for Piers 2 & 3 at W-N Ramp – Static Analysis

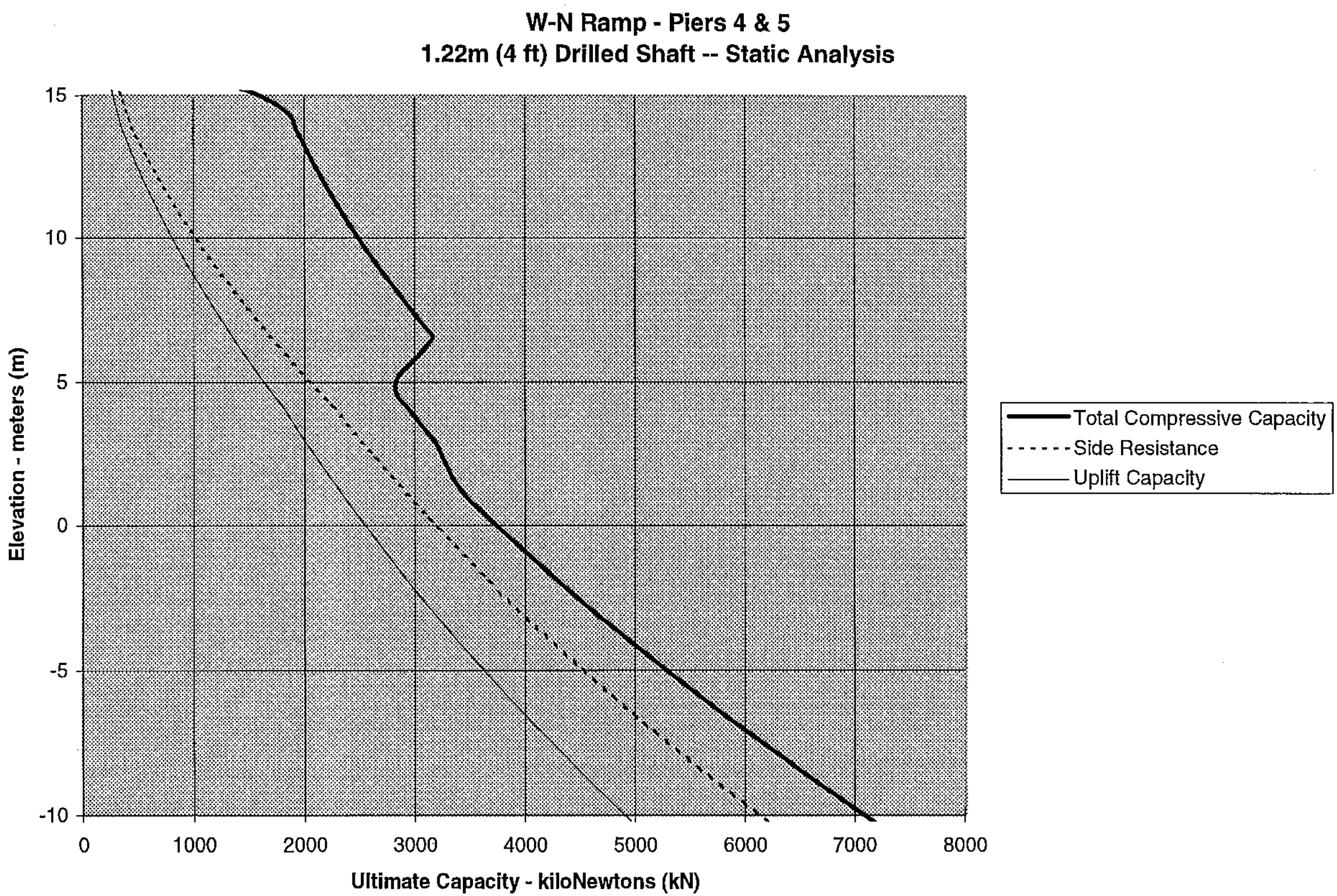


Figure 6-10. Ultimate Capacity of 1.22 m (4 ft) Drilled Shaft for Piers 4 & 5 at W-N Ramp – Static Analysis

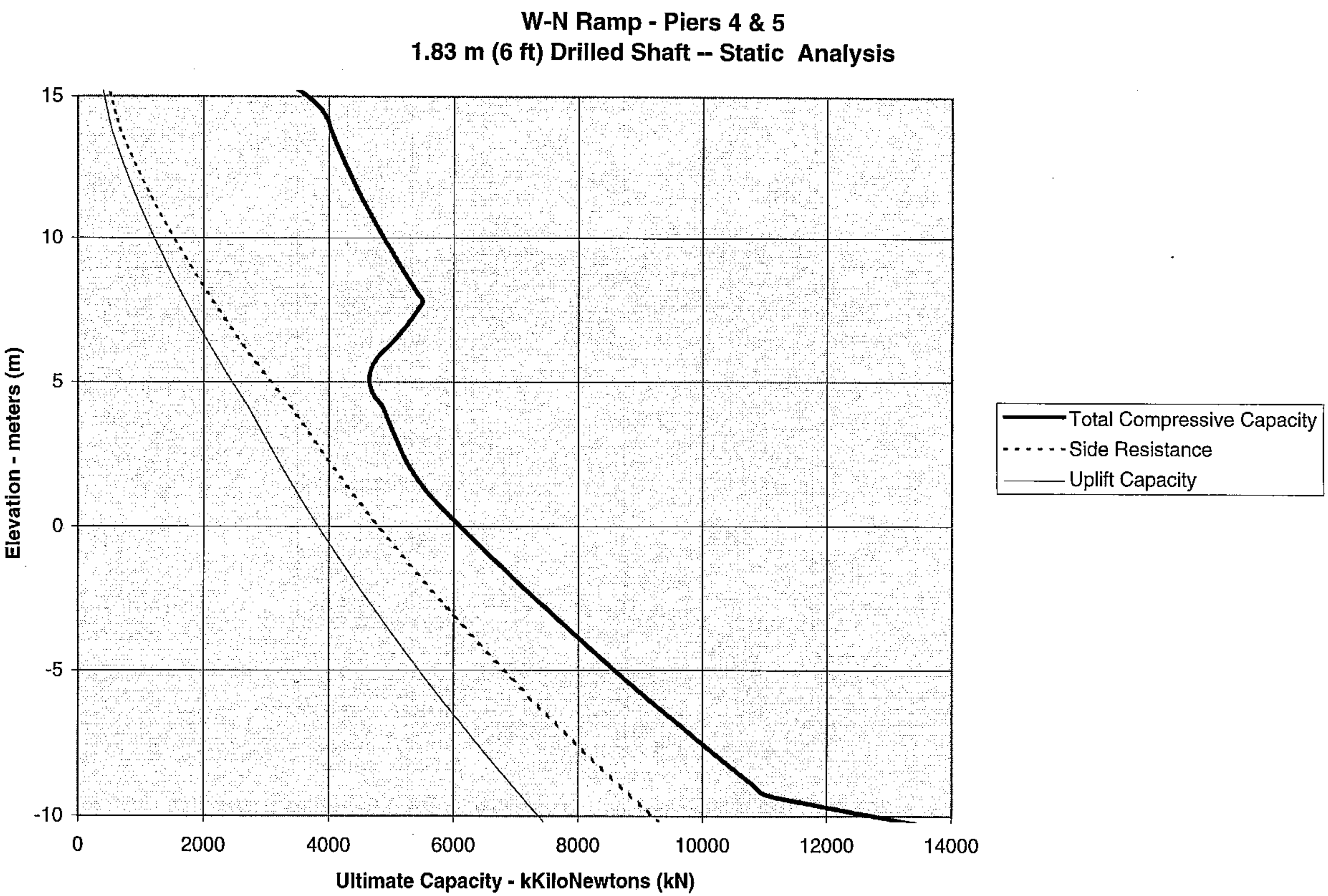


Figure 6-11. Ultimate Capacity of 1.83 m (6 ft) Drilled Shaft for Piers 4 & 5 at W-N Ramp – Static Analysis

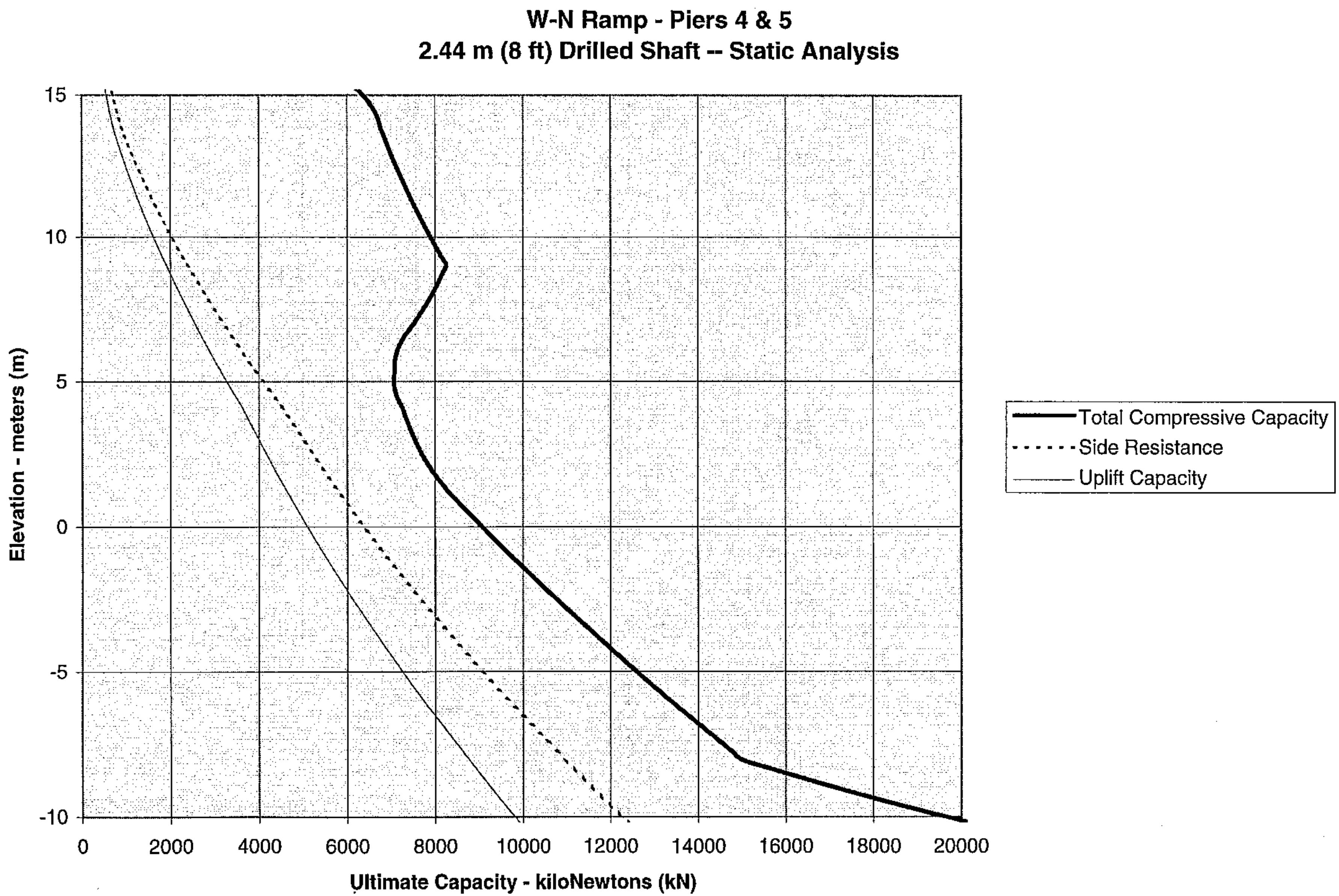


Figure 6-12. Ultimate Capacity of 2.44 m (8 ft) Drilled Shaft for Piers 4 & 5 at W-N Ramp -- Static Analysis

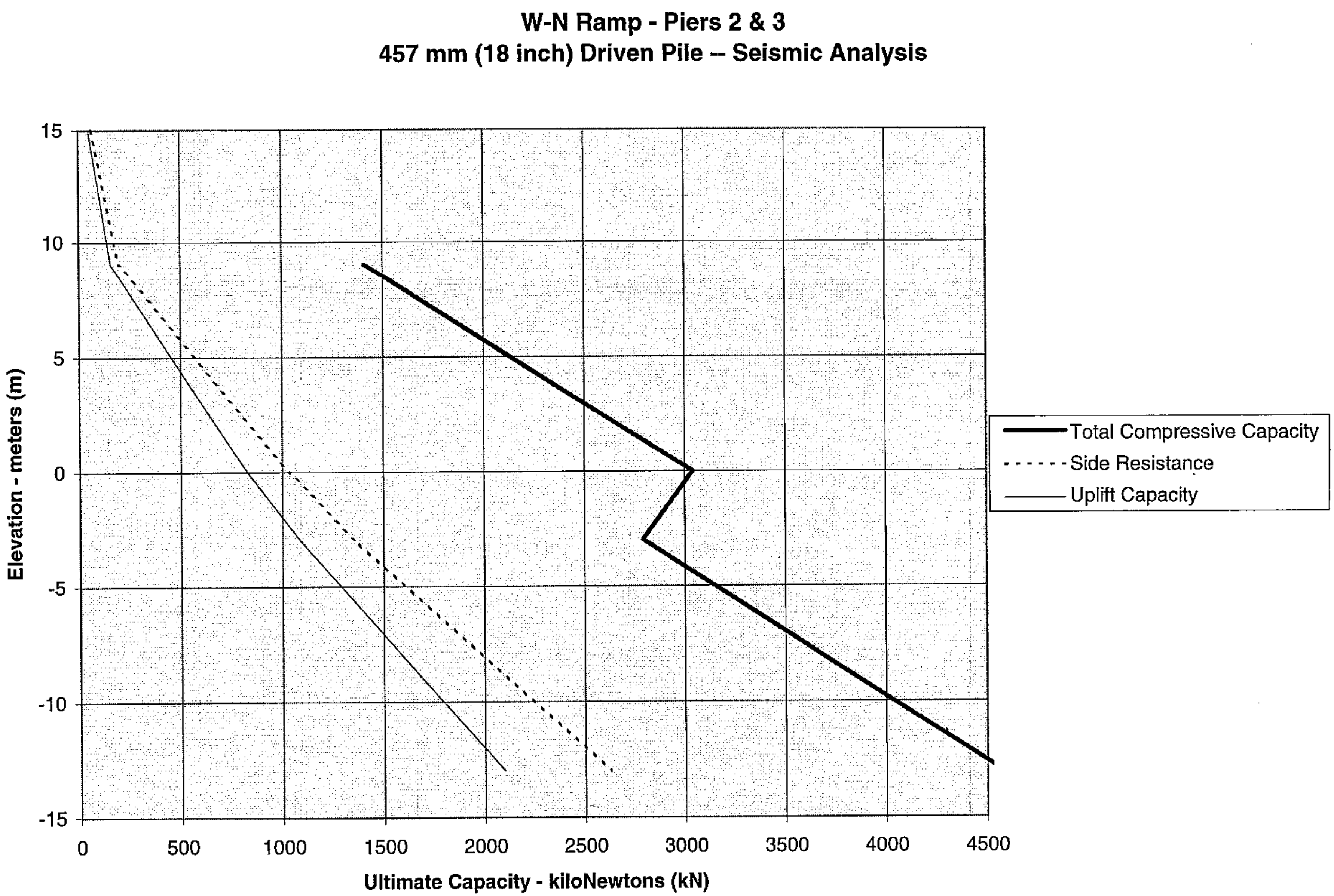


Figure 6-13. Ultimate Pile Capacity for 460 mm (18 inch) Driven, Closed-End Steel Piles for Piers 2 & 3 at W-N Ramp – Seismic Analysis

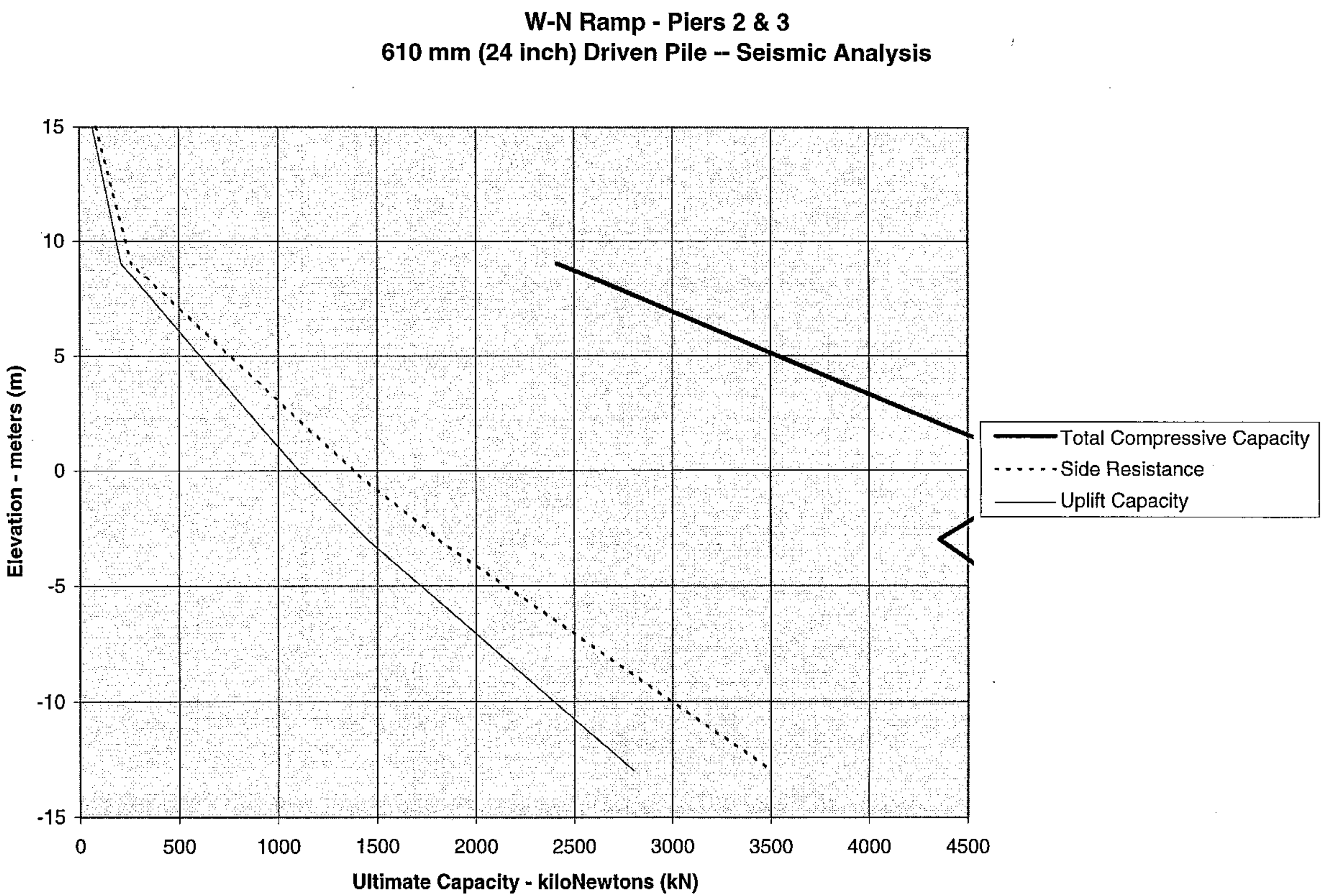


Figure 6-14. Ultimate Pile Capacity for 610 mm (24 inch) Driven, Closed-End Steel Piles for Piers 2 & 3 at W-N Ramp – Seismic Analysis

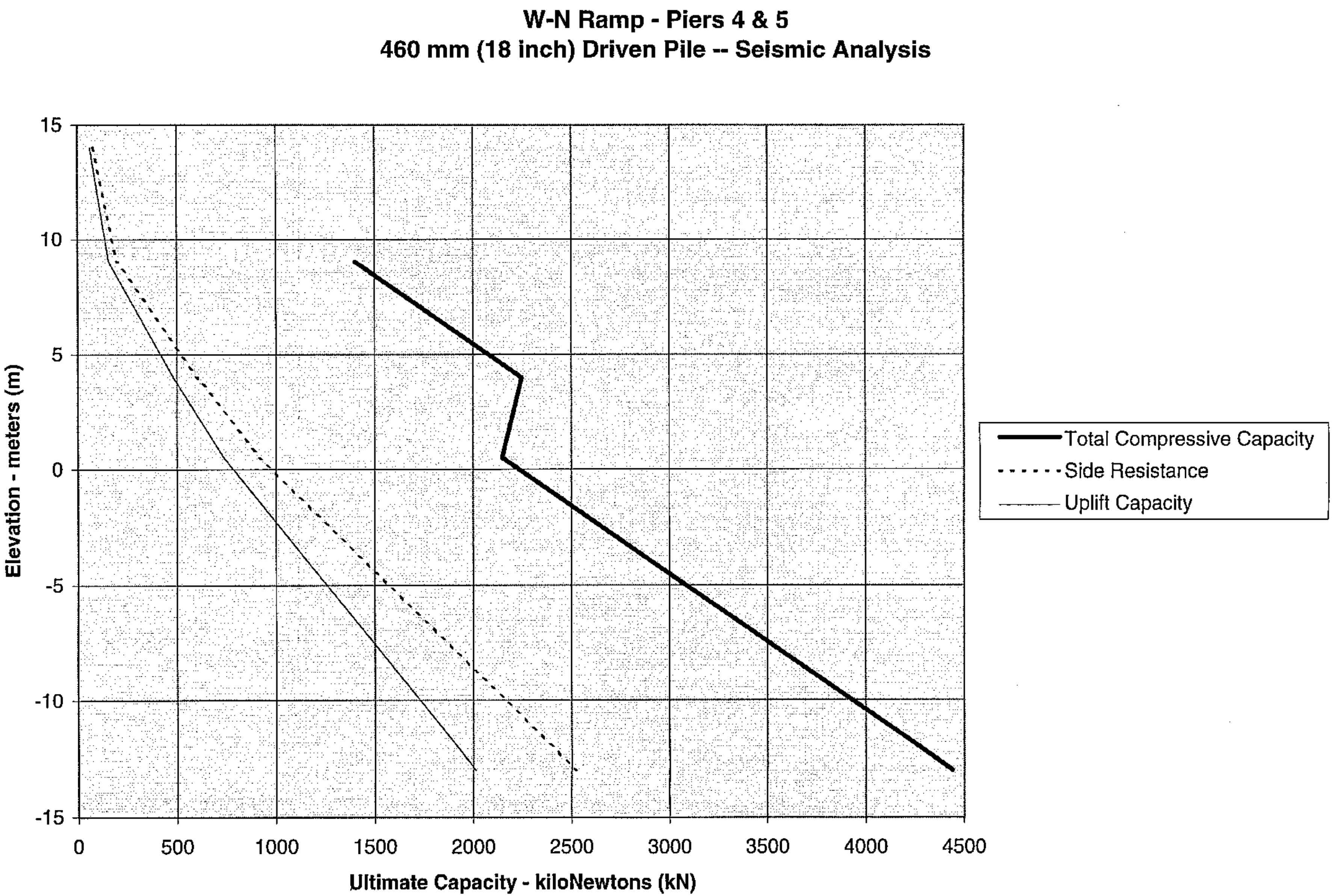


Figure 6-15. Ultimate Pile Capacity for 460 mm (18 inch) Driven, Closed-End Steel Piles for Piers 4 & 5 at W-N Ramp -- Seismic Analysis

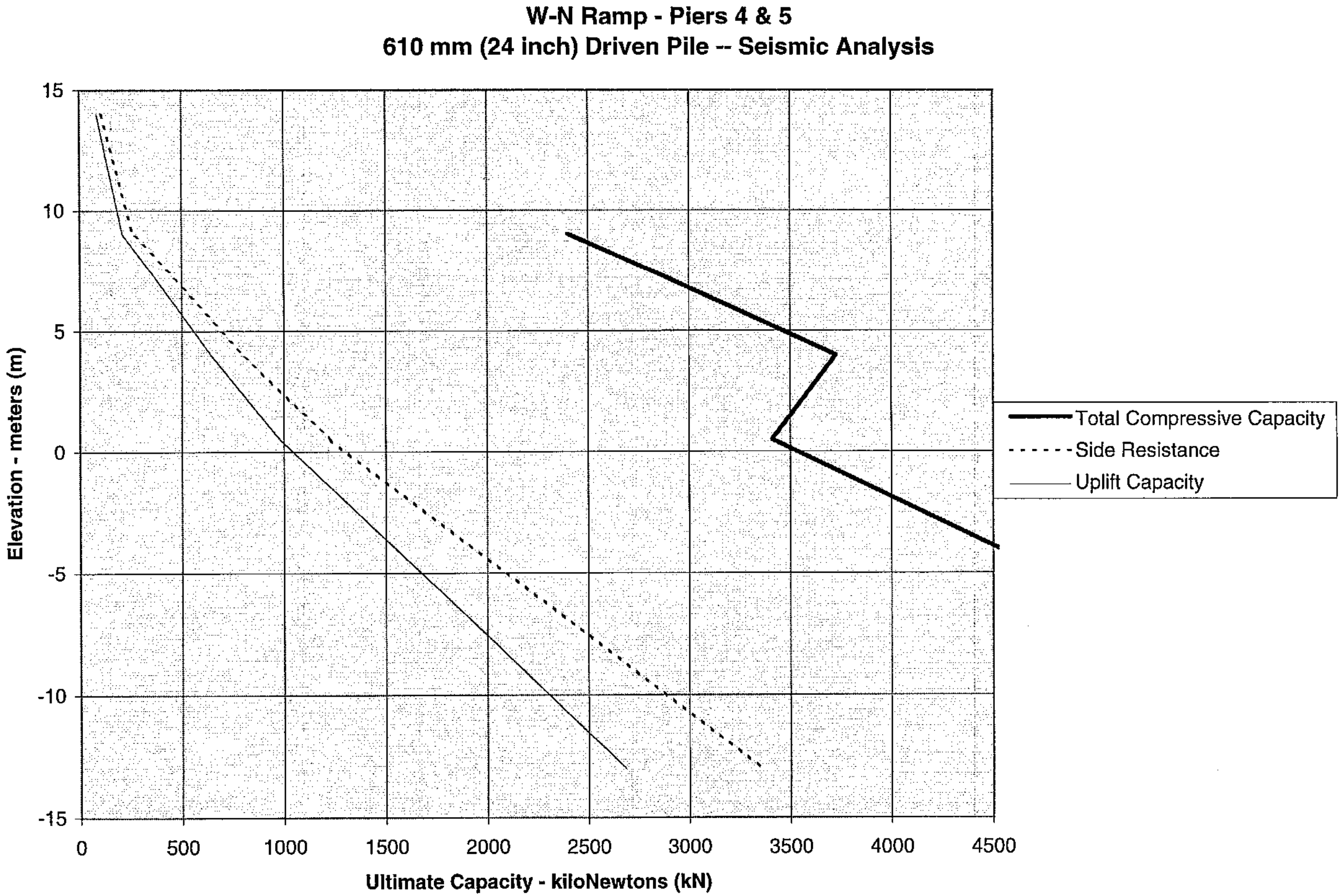


Figure 6-16. Ultimate Pile Capacity for 610 mm (24 inch) Driven, Closed-End Steel Piles for Piers 4 & 5 at W-N Ramp – Seismic Analysis

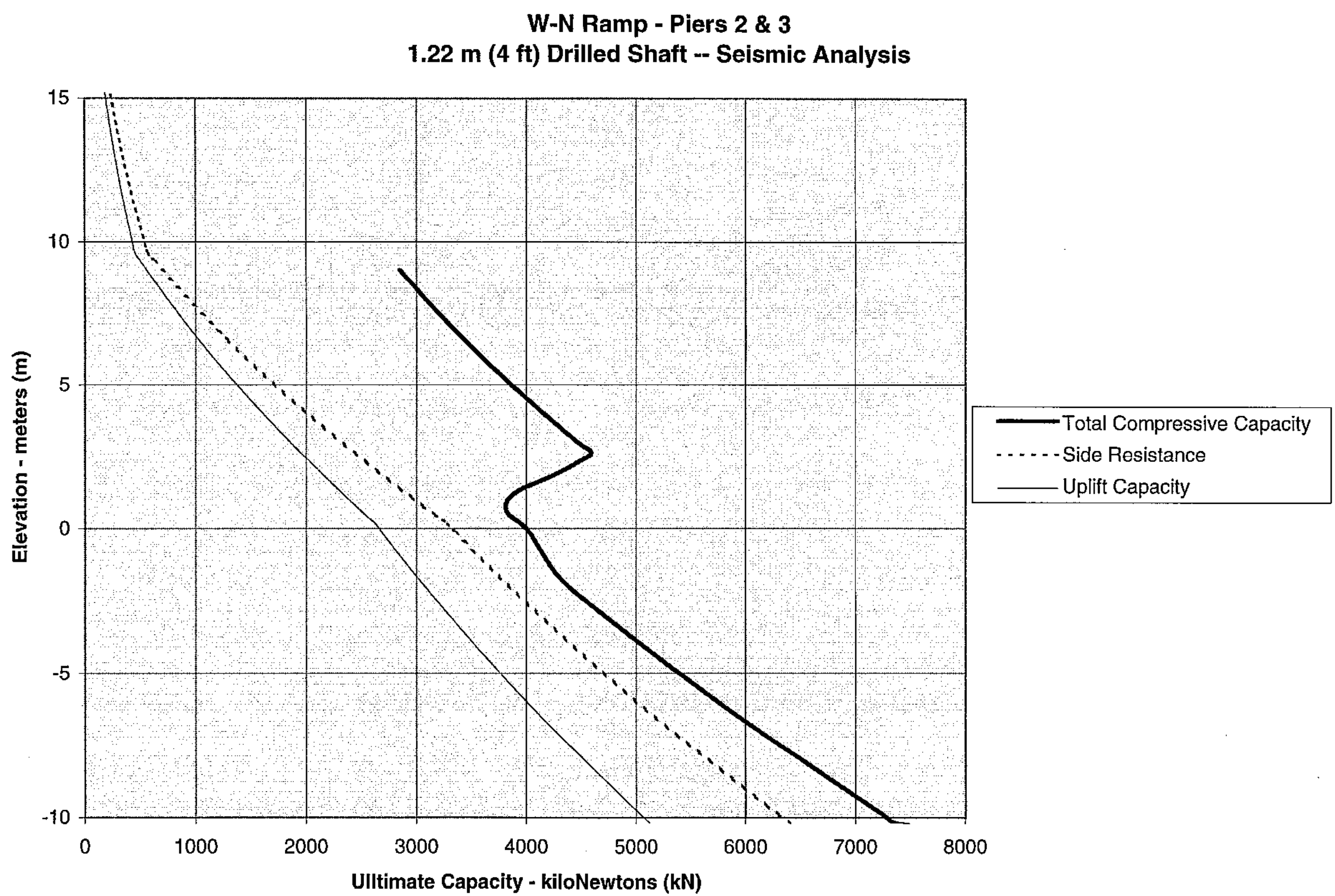


Figure 6-17. Ultimate Capacity of 1.22 m (4 ft) Drilled Shaft for Piers 2 & 3 at W-N Ramp – Static Analysis

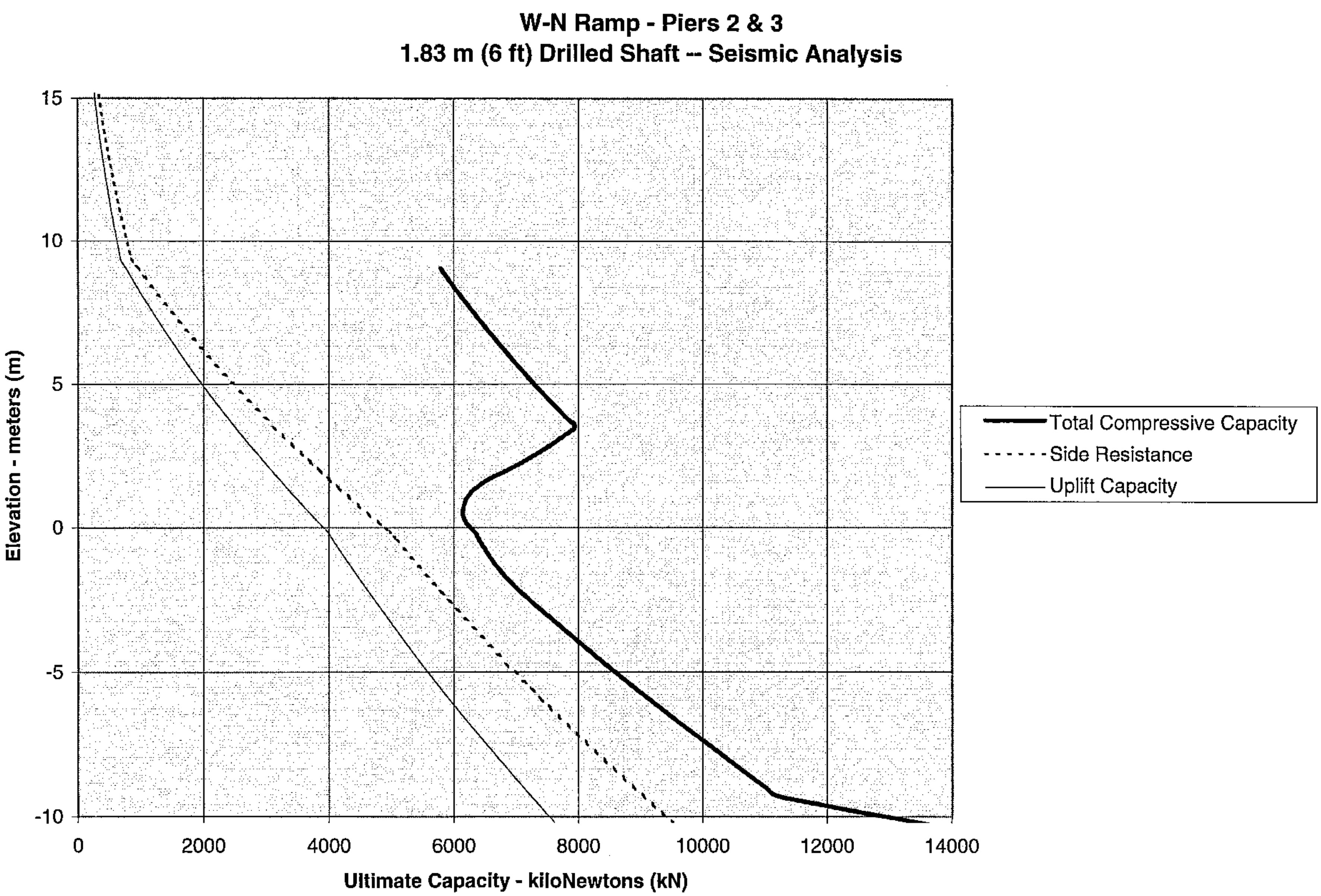


Figure 6-18. Ultimate Drilled Shaft Capacity of 1.83 m (6 ft) Drilled Shaft for Piers 2 & 3 at W-N Ramp – Seismic Analysis

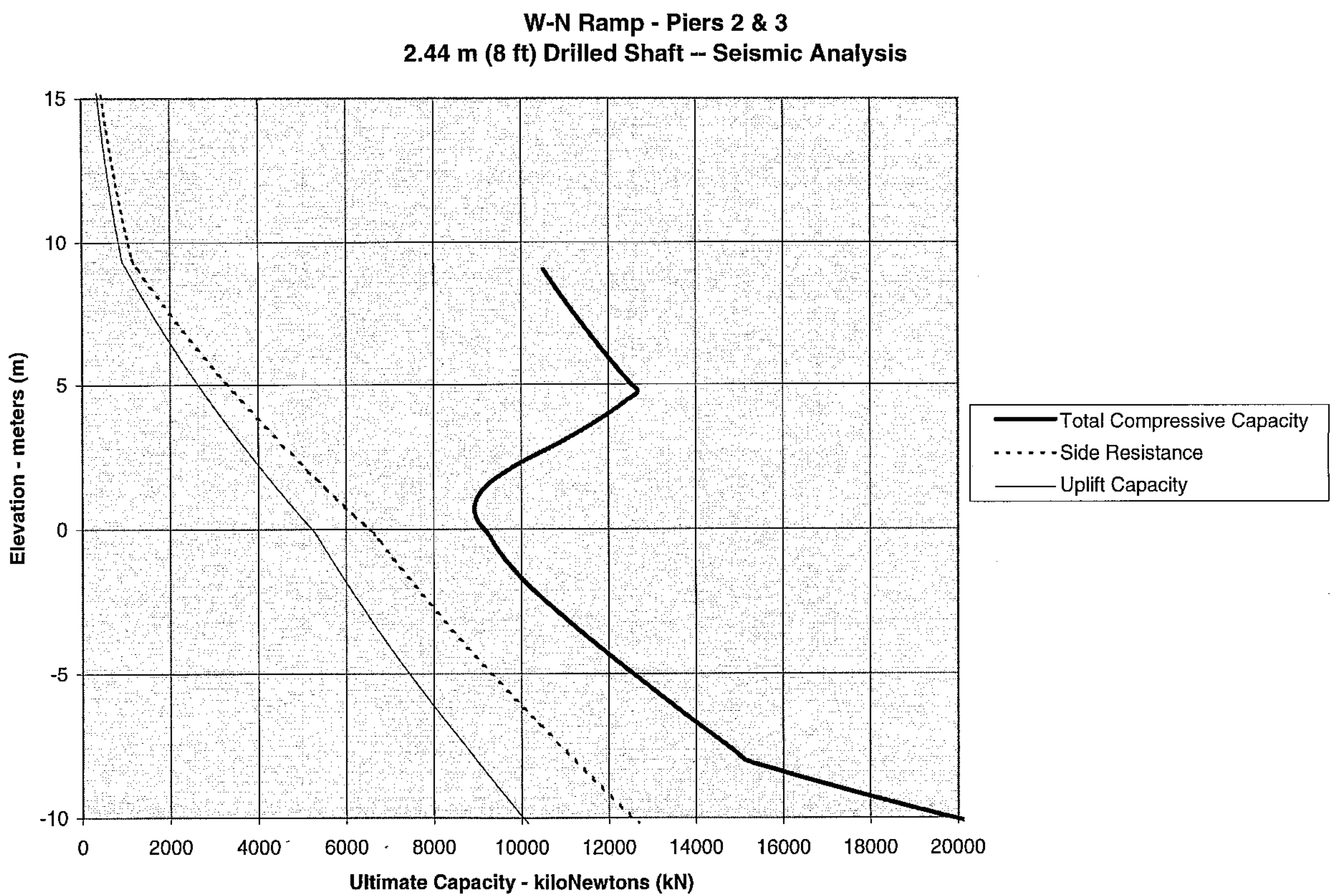


Figure 6-19. Ultimate Drilled Shaft Capacity of 2.44 m (8 ft) Drilled Shaft for Piers 2 & 3 at W-N Ramp – Seismic Analysis

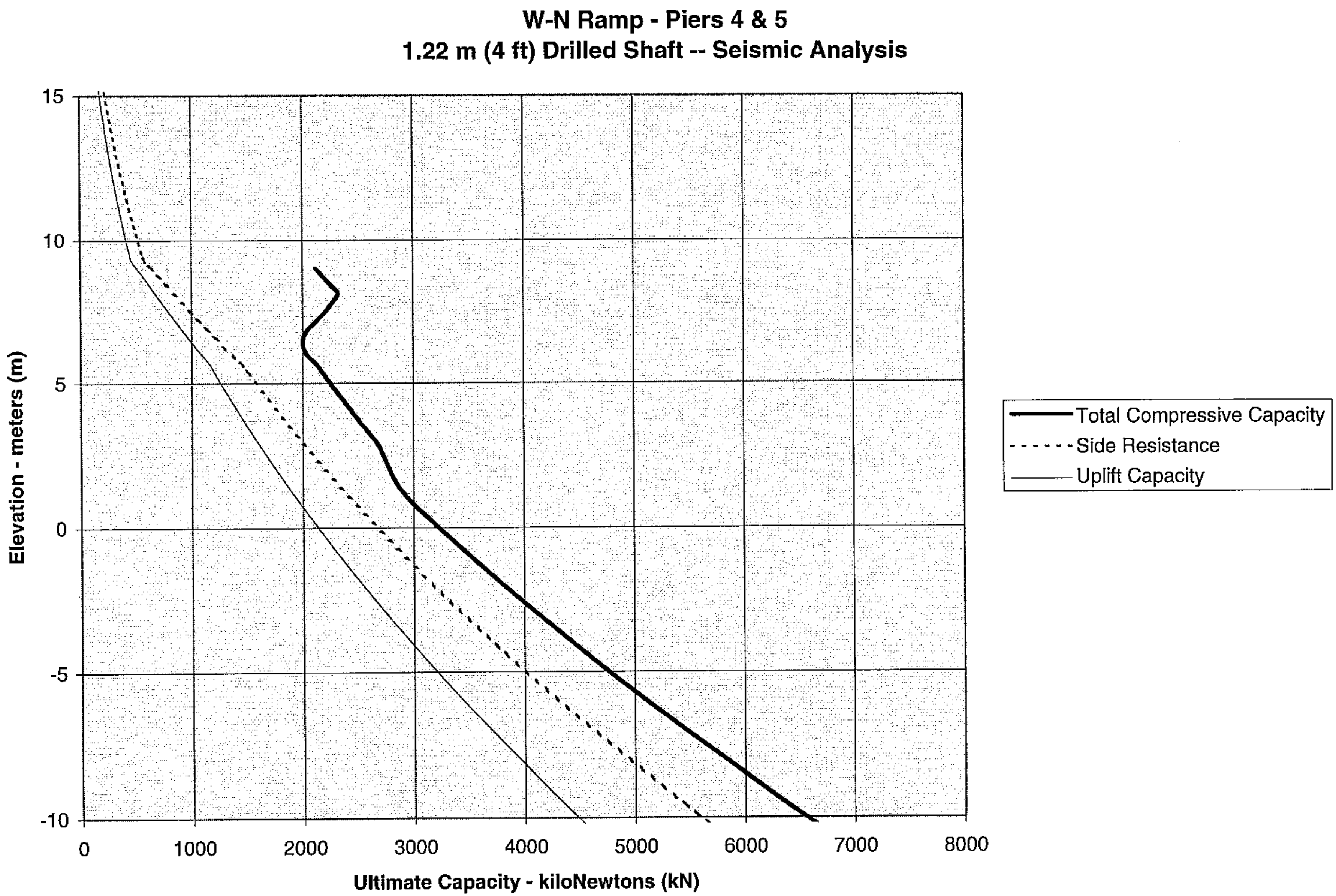


Figure 6-20. Ultimate Capacity of 1.22 m (4 ft) Drilled Shaft for Piers 4 & 5 at W-N Ramp – Seismic Analysis

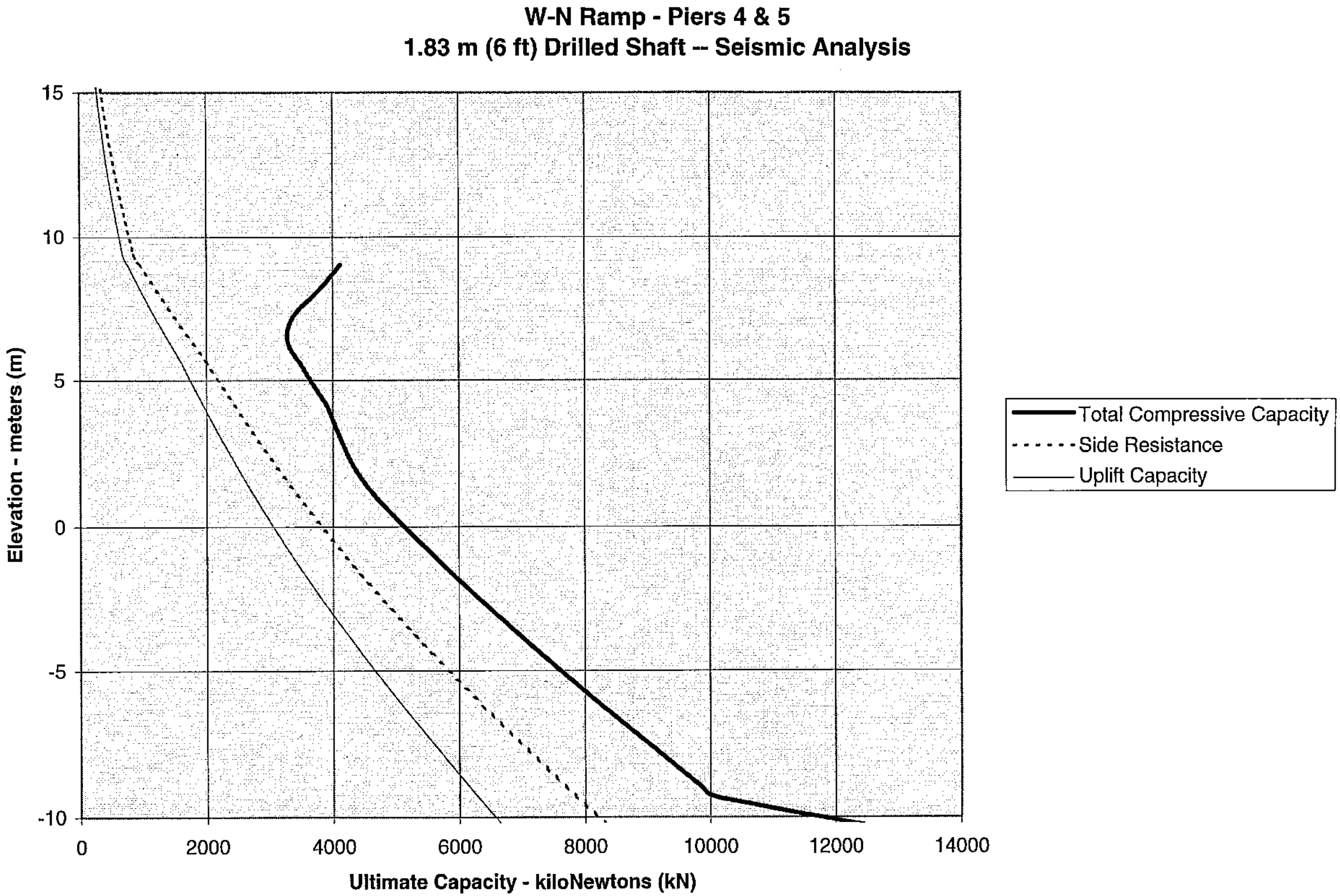


Figure 6-21. Ultimate Drilled Shaft Capacity of 1.83 m (6 ft) Drilled Shaft for Piers 4 & 5 at W-N Ramp – Seismic Analysis

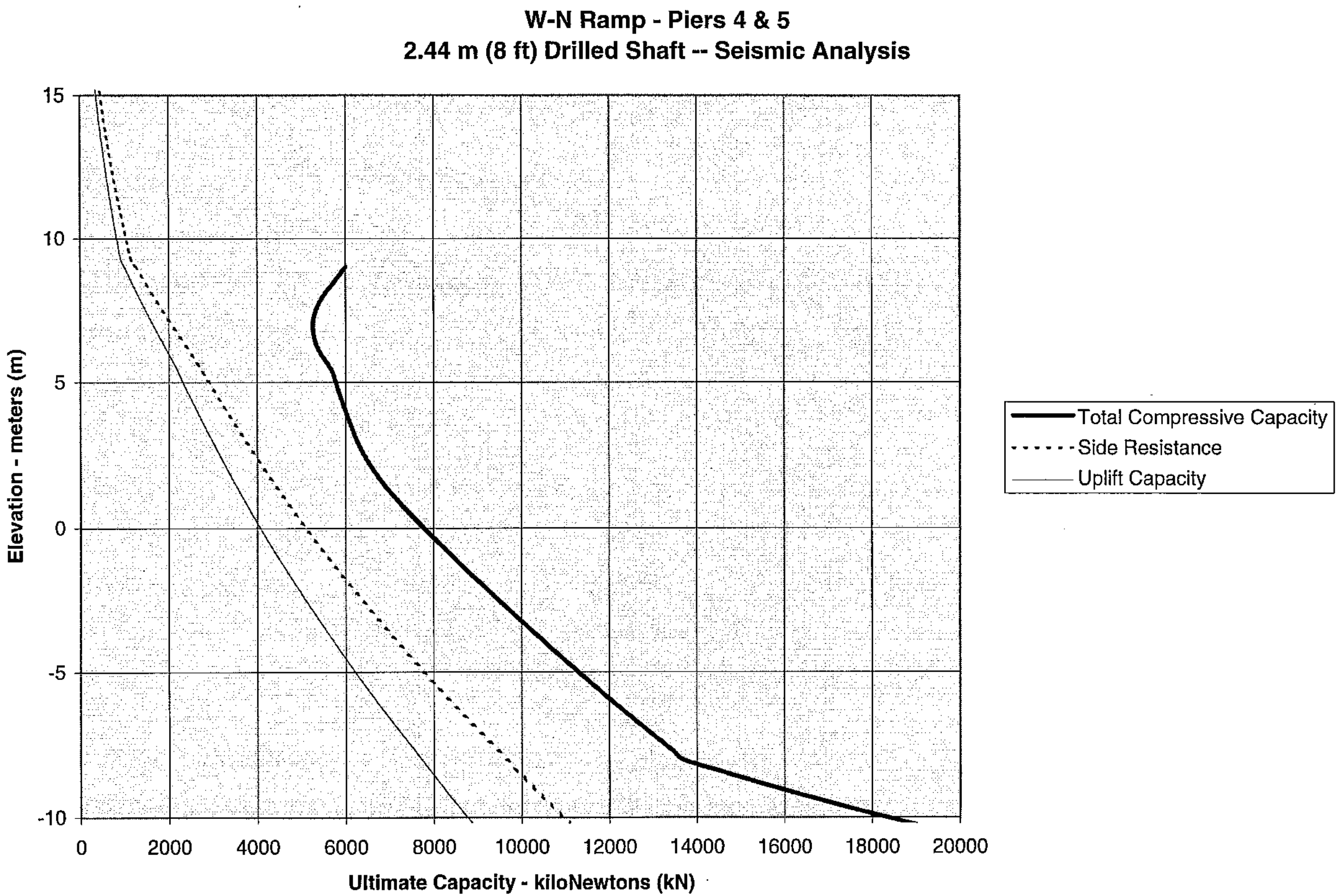


Figure 6-22. Ultimate Drilled Shaft Capacity of 2.44 m (8 ft) Drilled Shaft for Piers 4 & 5 at W-N Ramp – Seismic Analysis

1997 SOIL TEST HOLE LOGS

FOR

W-N RAMP

Proj. No.: 116184.G4

SOIL TEST HOLE LOG

Project: SR167, CS 1765/6, OL-2305 Drilling Contractor: WSDOT

Drilling Method & Equipment: CME 45 Skid Rig, Wash Rotary w/ 100 mm Casing Logger: M. Xue/CivilTech

Depth Below Surface (FT)	Sample: Interval	Sample: Number and Type	Sample: Recovery (FT)	Standard Penetration Test Results 6"-6"-6" (N)	Soil Description: Soil name, uscs group symbol, color, moisture content, relative density or consistency, soil structure, mineralogy	Comments: Depth of casing, drilling rate, drilling fluid loss, tests and instrumentation
Elevation: 28m (NAVD88)					Location: Sta. 9+35m, Offset 11m R of CL	**Test Hole H-8-97**
Start: 12/04/97					Finish: 12/05/97	Water Level:
						Sheet 1 of 2
						Started drilling at 10:16 am on 12/04/97
5.0	4.5 - 6.0	S-1	0.8	1-2-5	SILTY SAND, (SM), fine to coarse, mottle brown and gray, very moist, loose, with some fine to coarse gravel (FILL)	Driller notes scattered cobbles. Slow drilling from 6' due to scattered cobbles and gravels
10.0	9.5 - 10.0	S-2	0.8	7-6-17	SILT, (ML), brown mottled with gray and orange, very moist, stiff to very stiff, some coarse sand and fine to coarse angular gravel (FILL)	Sampler driven on gravel or cobble for last 6"
15.0	14.5 - 16.0	S-3	1.2	5-7-8	SAND, (SP), fine to medium, dark brown, very moist, medium dense, trace of gravel (FILL)	
20.0	19.5 - 21.0	S-4	0.8	17-14-16	SILTY SAND TO SILTY GRAVEL, (SM/GM), fine to coarse sand, dark gray, wet, medium dense to dense (FILL)	
25.0	24.5 - 26.0	S-5	0.0	5-1-2	NO RECOVERY, cuttings indicate that materials are silty sand and gravel (FILL)	Driller notes that break through dense gravel layer at 24.5', Driller notes 2' loose layer from 24.5' to 26.5', then drilling becomes slow.
30.0	29.5 - 31.0	S-6	1.0	34-29-24	SANDY GRAVEL, (GP) fine to coarse, brown to gray, wet, dense, some silt (FILL)	Stop at 33' at 4:15 pm Resume drilling at 9:10 am of 12/05/97
35.0	34.5 -	S-7	0.9	20-5-3	SILT, (ML), dark gray, very moist, medium	

NOTES:

1) Test hole located on south abutment 7.5' east and 2.5' north of southeast corner of bridge.
2) All blowcounts obtained with WSDOT automatic hammer
3) Water not measured. water in test hole from rotary wash drilling method.

1 foot = 0.3048 meters
1 inch = 25.4 millimeters

CH2M HILL

Proj. No.: 116184.G4

SOIL TEST HOLE LOG

Project: SR167, CS 1765/6, OL-2305 Drilling Contractor: WSDOT

Drilling Method & Equipment: CME 45 Skid Rig, Wash Rotary w/ 100 mm Casing Logger: M. Xue/CivilTech

Depth Below Surface (FT)	Sample: Interval	Sample: Number and Type	Sample: Recovery (FT)	Standard Penetration Test Results 6"-6"-6" (N)	Soil Description: Soil name, uscs group symbol, color, moisture content, relative density or consistency, soil structure, mineralogy	Comments: Depth of casing, drilling rate, drilling fluid loss, tests and instrumentation
Elevation: 28m (NAVD88)					Location: Sta. 9+35m, Offset 11m R of CL	**Test Hole H-8-97**
Start: 12/04/97					Finish: 12/05/97	Water Level:
						Sheet 2 of 2
	36.0				stiff to stiff, with clay layers and fine sand seams, traces of organics	Driller notes smooth drilling from 37', indicating silt and sand materials
40.0	39.5 - 41.0	S-8	1.2	2-3-4	SANDY SILT, (ML), dark gray, very moist, medium stiff to stiff, very fine sand, traces of gravel and wood chips	Driller notes sandy material with traces of gravel from about 42'.
45.0	44.5 - 46.0	S-9	1.3	10-13-16	SAND, (SP), fine to medium, dark gray to black, very moist, medium dense to dense.	
					END OF SOIL TEST HOLE AT 46'	Stopped drilling at 11:20 am on 12/05/97
50.0						
55.0						
60.0						
65.0						
70.0						

NOTES:

Installed piezometer. Total length = 43'. Bottom 2' solid casing (1"). 5' of screen with 1/32" slot at 1/4" spacing. Sand pack located in bottom 13'. Top 36' of piezometer 1" solid pvc casing. Top 36' backfilled with bentonite. Locking cap located at the ground surface.

1 foot = 0.3048 meters
1 inch = 25.4 millimeters

CH2M HILL

Proj. No.: 116184.G4

SOIL TEST HOLE LOG

Project: SR-167, CS 1765/6, OL-2305 Drilling Contractor: WSDOT

Drilling Method & Equipment: Longyear BK-80 Truck Mounted Rig Logger: M. Xue/CivilTech

Depth Below Surface (FT)	Sample: Interval	Sample: Number and Type	Sample: Recovery (FT)	Standard Penetration Test Results 6"-6"-6" (N)	Soil Description: Soil name, uscs group symbol, color, moisture content, relative density or consistency, soil structure, mineralogy	Comments: Depth of casing, drilling rate, drilling fluid loss, tests and instrumentation
Elevation: 23 m					Location: Sta. 9+48m; Offset 9.8m R of CL	**Test Hole H-9-97**
Start: 12/17/97					Finish: 12/23/97	Water Level: 21m
						Sheet 1 of 4
						Start drilling at 8:20 am on 12/17/97. Began drilling with 6" HSA.
5.0	4.0 - 5.5	S-1	0.2	4-5-8	SILTY SAND, (SM), fine to medium, brown, very moist, traces of rounded gravel	
10.0	9.0 - 10.5	S-2	0.6	3-2-4	SILTY SAND TO SANDY SILT, (SM/ML), medium to coarse, very moist, loose to medium dense, some gravel	
15.0	14.0 - 15.5	S-3	0.6	2-11-6	SILTY SAND,(SM), medium to coarse, brown, moist, medium dense, some subrounded gravel	
20.0	19.0 - 20.5	S-4	1.1	2-2-2	SILT,(ML), dark gray, moist, soft to medium stiff, some very fine sand, traces of organics	8" silty layer at bottom of sample
25.0	24.0 - 25.5	S-5	1.5	6-7-7	SILTY SAND, (SM), very fine, dark gray, wet, medium dense	
30.0	29.0 - 30.5	S-6	1.0	6-9-13	SAND, (SP), medium to coarse, dark gray to black, wet, medium dense, traces of silt	Set-up for rotary wash after observing heave in auger
35.0	34.0	S-7	1.0	10-11-13	SAND, (SP), medium to coarse, dark gray	

NOTES:

1) Test hole located below N-W Ramp bridge 19' east of bridge column and 5' north of guard rail on SR-18
2) All blowcounts obtained with WSDOT automatic hammer

1 foot = 0.3048 meters
1 inch = 25.4 millimeters

CH2M HILL

Proj. No.: 116184.G4

SOIL TEST HOLE LOG

Project: SR-167, CS 1765/6, OL-2305 Drilling Contractor: WSDOT

Drilling Method & Equipment: Longyear BK-80 Truck Mounted Rig Logger: M. Xue/CivilTech

Depth Below Surface (FT)	Sample Interval	Sample Number and Type	Sample Recovery (FT)	Standard Penetration Test Results 6"-6"-6" (N)	Soil Description: Soil name, uscs group symbol, color, moisture content, relative density or consistency, soil structure, mineralogy	Comments: Depth of casing, drilling rate, drilling fluid loss, tests and instrumentation
Elevation: 23m					Location: Sta. 9+48m; Offset 9.8m R of CL	Test Hole H-9-97
Start: 12/17/97					Finish: 12/23/97	Water Level: 21m
						Sheet 2 of 4
	35.5				to black, moist, medium dense, traces of gravel	
40.0	39.0 - 40.0	S-8	1.0	7-5-8	SAND, (SP,) fine to coarse, dark gray very moist, medium dense, trace of silt and gravel	Sand inside auger. Washed out and then sampled
45.0	44.0 - 45.5	S-9	0.9	7-11-13	SAND, (SP) fine to coarse, dark gray, very moist, medium dense, traces of silt and some subrounded gravel	
50.0	49.0 - 50.5	S-10	1.5	3-6-5	SAND, (SP), fine to coarse, dark gray to black, very moist, loose to medium dense, with some gravel and wood chips (2")	Recovered 2" wood chip in sampler
55.0	54.0 - 55.5	S-11	0.5	3-4-10	GRAVELLY SAND TO SANDY GRAVEL, (SP/GP), coarse sand with medium to coarse gravel, dark gray to black, wet, medium dense	Plug stuck, pulled out. Washed auger and took sample
60.0	59.0 - 60.5	S-12	0.6	2-5-20	SAND AND GRAVEL (SP/GP), coarse sand with medium to coarse gravel, dark gray to black, very moist, medium dense, traces of silt	2' sand inside auger. Washed through rod. Switched to HQ wireline
65.0	64.0 - 65.5	S-13	0.8	9-8-8	SAND, (SP), fine to coarse, black, very moist, medium dense, with silt	
70.0	69.0 -	S-14	0.9	10-11-13	SILTY SAND, (SM), fine, gray, very moist	

NOTES:

1 foot = 0.3048 meters
1 inch = 25.4 millimeters

CH2M HILL

Proj. No.: 116184.G4

SOIL TEST HOLE LOG

Project: SR-167, CS 1765/6, OL-2305 Drilling Contractor: WSDOT

Drilling Method & Equipment: Longyear BK-80 Truck Mounted Rig Logger: M. Xue/CivilTech

Depth Below Surface (FT)	Sample: Interval	Sample: Number and Type	Sample: Recovery (FT)	Standard Penetration Test Results 6"-6"-6" (N)	Soil Description: Soil name, uscs group symbol, color, moisture content, relative density or consistency, soil structure, mineralogy	Comments: Depth of casing, drilling rate, drilling fluid loss, tests and instrumentation
Elevation: 23m					Location: Sta. 9+48m; Offset 9.8m R of CL	**Test Hole H-9-97**
Start: 12/17/97					Finish: 12/23/97	Water Level: 21m
						Sheet 3 of 4
	70.5				to wet, medium dense, uniformly graded	
75.0	74.0 - 75.5	S-15	1.5	2-5-4	CLAYEY SILT, (ML/MH), dark gray, very moist, stiff, with fine sandy silt layer	
80.0	79.0 - 80.5	S-16	1.5	4-3-4	SILTY SAND, (SM), coarse, gray to dark gray, wet, loose to medium dense	Upper 5" of soil in sampler appears to be gray silty clay
85.0	84.0 - 85.5	S-17	0.1	3-4-7	SILTY SAND, (SM), coarse, gray to dark gray, very moist, medium dense, some angular gravel	Stopped drilling at 3:35 pm Resume drilling at 12:12 pm of 12/22/97. Driller notes gravel from 81' to 84'
90.0	89.0 - 90.5	S-18	1.0	3-3-4	SILTY SAND TO SILTY GRAVEL, (SM/GM), coarse sand and fine to medium gravel, gray, very moist, loose, subangular gravel	
95.0	94.0 - 95.5	S-19	0.0	4-3-4	NO RECOVERY	
100.0	99.0 - 100.5	S-20	1.4	2-3-4	SILTY SAND TO SILTY GRAVEL, (SM/GM), coarse sand and fine to medium gravel, gray, very moist, loose	Over drive sample for better recovery
105.0	104.0 -	S-21	1.3	2-6-5	SILTY SAND TO SILTY GRAVEL, (SM/	Over drive 3"

NOTES:

1 foot = 0.3048 meters
1 inch = 25.4 millimeters

CH2M HILL

Proj. No.: 116184.G4

SOIL TEST HOLE LOG

Project: SR-167, CS 1765/6, OL-2305 Drilling Contractor: WSDOT

Drilling Method & Equipment: Longyear BK-80 Truck Mounted Rig Logger: M. Xue/CivilTech

Elevation: 23m | Location: Sta. 9+48m; Offset 9.8m R of CL | **Test Hole H-9-97**

Start: 12/17/97 | Finish: 12/23/97 | Water Level: 21m

Depth Below Surface (FT)	Sample: Interval	Sample: Number and Type	Sample: Recovery (FT)	Standard Penetration Test Results 6"-6"-6" (N)	Soil Description: Soil name, uscs group symbol, color, moisture content, relative density or consistency, soil structure, mineralogy	Comments: Depth of casing, drilling rate, drilling fluid loss, tests and instrumentation
	105.5				GM), coarse sand and gravel, gray, very moist, medium dense	
110.0	109.0 - 110.5	S-21	1.5	2-3-4	SILTY SAND TO SILTY GRAVEL, (SM/GM), coarse sand and fine to medium gravel, gray, very moist, loose	Over drive 5" Sub-rounded to subangular gravel
115.0	114.0 - 115.5	S-22	0.9	3-3-4	SILTY SAND TO SILTY GRAVEL (SM/GM), similar to S-21	
					END OF SOIL TEST HOLE AT 115.5 FEET	Stopped drilling at 2:50 pm on 12/23/97
120.0						
125.0						
130.0						
135.0						
140.0						

NOTES:

1 foot = 0.3048 meters
1 inch = 25.4 millimeters

Proj. No.: 116184.G4

SOIL TEST LOG

Project: SR-167, CS 1765/6, OL-2305 Drilling Contractor: WSDOT

Drilling Method & Equipment: Longyear BK-80 Truck Mounted Rig Logger: M. Xue/CivilTech

Depth Below Surface (FT)	Sample: Interval	Sample: Number and Type	Sample: Recovery (FT)	Standard Penetration Test Results 6"-6"-6" (N)	Soil Description: Soil name, uscs group symbol, color, moisture content, relative density or consistency, soil structure, mineralogy	Comments: Depth of casing, drilling rate, drilling fluid loss, tests and instrumentation
Elevation: 24.5m					Location: Sta. 10+03m; Offset 9.1m R of CL	**Test Hole H-10-97**
Start: 12/23/97					Finish: 12/23/97	Water Level: Not Measured
						Sheet 1 of 3
						started drilling at 9:00 am on 12/23/97 using 6" HAS
5.0	4.0 - 5.5	S-1	0.4	4-2-3	SILTY SAND, (SM), coarse, brown, moist, loose, some subrounded gravels, some organics	
10.0	9.0 - 10.5	S-2	0.6	1-2-3	SAND, (SP), fine, dark gray, moist, loose, with trace of silt	3" thick layer of gray silt with trace of sand at top of sample
15.0	14.0 - 15.5	S-3	1.5	1-2-3	SILTY SAND TO SANDY SILT, (SM/ML), very fine, dark gray, very moist, loose, soft	
20.0	19.0 - 20.5	S-4	1.2	2-3-3	SAND TO SILTY SAND, (SP/SM), very fine, dark gray, very moist to wet, loose	1" thick layer of silt
25.0	24.5 - 25.5	S-5	1.2	3-3-6	SAND, (SP) fine to medium, dark gray to black, wet, medium dense, some silt and gravel	Bottom heaved, Washed before sampled
30.0	29.0 - 30.5	S-6	1.1	3-6-6	SAND, (SP), fine to medium, dark gray to black, wet, medium dense, some silt and trace of subangular gravel	3' heave. One tap before sampling. No washing
35.0	34.0 -	S-7	0.6	3-5-6	SAND, (SP), medium to coarse, dark gray	Bottom heave. Washed

NOTES:

1) Test hole located below N-W Ramp bridge 15' east and 5' south of bridge column on SR-18
2) All blowcounts obtained with WSDOT automatic hammer

1 foot = 0.3048 meters
1 inch = 25.4 millimeters

CH2M HILL

Proj. No.: 116184.G4

SOIL TEST LOG

Project: SR-167, CS 1765/6, OL-2305 Drilling Contractor: WSDOT

Drilling Method & Equipment: Longyear BK-80 Truck Mounted Rig Logger: M. Xue/CivilTech

Depth Below Surface (FT)	Sample: Interval	Sample: Number and Type	Sample: Recovery (FT)	Standard Penetration Test Results 6"-6"-6" (N)	Soil Description: Soil name, uscs group symbol, color, moisture content, relative density or consistency, soil structure, mineralogy	Comments: Depth of casing, drilling rate, drilling fluid loss, tests and instrumentation
Elevation: 24.5m					Location: Sta. 10+03m; Offset 9.1m R of CL	Test Hole H-10-97
Start: 12/23/97					Finish: 12/23/97	Water Level: Not measured
						Sheet 2 of 3
	35.5				to black, wet, medium dense, with some fine subangular to subrounded gravels	before sampling
40.0	39.0 - 40.5	S-8	0.8	2-1-5	SAND, (SP), coarse. black, wet, loose, with fine to coarse rounded to subrounded gravel	Bottom heave, Washed before sampling
45.0	44.0 - 45.5	S-9	1.0	1-1-6	SAND, (SP), medium to coarse, black, very moist, loose, with some rounded gravel and trace of silt	Over drove sample 6" for better recovery
50.0	49.0 - 50.5	S-10	1.0	10-14-8	SAND AND GRAVEL, (SP/GP), fine to coarse sand, medium to coarse gravel, black, very moist, medium dense	Taped sampler bottom
55.0	54.0 - 55.5	S-11	1.5	2-2-11	SANDY SILT TO SILT, (ML), dark gray to gray, very moist, stiff	
60.0	59.0 - 60.5	S-12	1.2	5-6-11	SAND TO SILTY SAND, (SP/SM), fine to coarse, dark gray to black, very moist, medium dense	
65.0	64.5 - 65.5	S-13	1.5	8-32-41	SANDY GRAVEL, (GP), fine to coarse subrounded gravel, coarse sand, dark gray, moist, very dense, with some silt	Top 8" is black coarse sand
70.0	69.0 -	S-14	1.5	5-4-3	SILTY SAND, (SM), fine to coarse, dark	5" layer of silt in sand

NOTES:

1 foot = 0.3048 meters
1 inch = 25.4 millimeters

CH2M HILL

Proj. No.: 116184.G4

SOIL TEST LOG

Project: SR-167, CS 1765/6, OL-2305 Drilling Contractor: WSDOT

Drilling Method & Equipment: Longyear BK-80 Truck Mounted Rig Logger: M. Xue/CivilTech

Depth Below Surface (FT)	Sample: Interval	Sample: Number and Type	Sample: Recovery (FT)	Standard Penetration Test Results 6"-6"-6" (N)	Soil Description: Soil name, uscs group symbol, color, moisture content, relative density or consistency, soil structure, mineralogy	Comments: Depth of casing, drilling rate, drilling fluid loss, tests and instrumentation
Elevation: 24.5M					Location: Sta. 10+03m; Offset 9.1m R of CL	**Test Hole H-10-97**
Start: 12/23/97					Finish: 12/23/97	Water Level: Not Measured
						Sheet 3 of 3
	70.5				gray, very moist, loose, some gravel	
75.0	74.0 - 75.5	S-15	1.2	17-3-4	SAND, (SP), fine to medium, dark gray, very moist, loose, some angular gravel (upper 8") over SANDY SILT, (ML), dark gray, very moist, soft (lower 12")	High blowcounts for first 6" possibly driving on gravel or cobble
80.0	79.0 - 80.5	S-16	1.3	28-7-4	SILTY SAND, (SM), fine to coarse, dark gray, very moist, loose to medium dense, some subrounded gravel	Possibly driving on gravel
					END OF TEST HOLE AT 80.5 FEET	Stopped drilling at 1:50 pm on 12./23/97
85.0						
90.0						
95.0						
100.0						
105.0						

NOTES:

1 foot = 0.3048 meters
1 inch = 25.4 millimeters

Proj. No.: 116184.G4

SOIL TEST LOG

Project: SR167, CS 1765/6, OL-2305   Drilling Contractor: WSDOT

Drilling Method & Equipment: CME 45 Skid Rig, Wash Rotary w/ 100 mm Casing   Logger: M. Xue/CivilTech

Depth Below Surface (FT)	Sample: Interval	Sample: Number and Type	Sample: Recovery (FT)	Standard Penetration Test Results 6"-6"-6" (N)	Soil Description: Soil name, uscs group symbol, color, moisture content, relative density or consistency, soil structure, mineralogy	Comments: Depth of casing, drilling rate, drilling fluid loss, tests and instrumentation
Elevation: 29m					Location: Sta. 10+38m, Offset 9.5m R of CL	**Test Hole H-11-97**
Start: 11/21/97					Finish: 11/21/97	Water Level: Not Measured
						Sheet 1 of 2
						Started drilling at 8:50 am on 11/21/97
5.0	5.0 - 6.5	S-1	0.5	3-7-8	SAND, (SP), coarse, dark brown, wet, medium dense, some silt and round gravel to 1" (FILL)	Blowcounts with WSDOT automatic hammer
10.0	10.0 - 11.5	S-2	0.0	7-12-16	NO RECOVERY	Two pieces of gravel droppedout of shoe. Blowcountswith WSDOT's automatic hammer
15.0	15.0 - 16.5	S-3	1.2	9-13-16	SAND, (SP), medium, brown, very moist to wet, medium dense, some gravel and gray silt (FILL)	Switched to safety hammer with rope cathead due to hydraulic leak with automatic hammer
20.0	20.0 - 21.5	S-4	0.9	15-19-20	SILTY SAND, (SP/SM), medium, mottled brown and gray, wet, dense, with silt and rounded to angular gravel (FILL)	Safety hammer with rope cathead procedure for blowcounts
25.0	25.0 - 26.5	S-5	0.5	16-11-8	SANDY SILT, (ML), mottle gray with orange, very moist, stiff to very stiff, some gravel (FILL)	Had difficulty pulling bit out.
						Driller notes more gravels from 28'. Abundant gravels at 29'
30.0	30.0 - 31.5	S-6	0.8	15-23-19	SILTY SAND, (SM), gray, very moist, dense, with rounded and angular gravel (FILL)	
						Driller notes less silt and more sand from 34'
35.0	35.0 -	S-7	0.4	10-10-11	SANDY GRAVEL, (GP/GM), dark gray,	

NOTES:

1) Test hole located on north abutment approximately 11' east and 4' north of northeast corner of bridge.
2) Blowcounts above 11.5' obtained with WSDOT's automatic hammer. All blowcounts below 11.5' obtained with safety hammer and rope cathead
3) Water not measured. Water in test hole from wash drilling method.

1 foot = 0.3048 meters
1 inch = 25.4 millimeters

Proj. No.: 116184.G4

SOIL TEST LOG

Project: SR167, CS 1765/6, OL-2305   Drilling Contractor: WSDOT

Drilling Method & Equipment: CME 45 Skid Rig, Wash Rotary w/ 100 mm Casing   Logger: M. Xue/CivilTech

Depth Below Surface (FT)	Sample			Standard Penetration Test Results 6"-6"-6" (N)	Soil Description	Comments
	Interval	Number and Type	Recovery (FT)		Soil name, uscs group symbol, color, moisture content, relative density or consistency, soil structure, mineralogy	Depth of casing, drilling rate, drilling fluid loss, tests and instrumentation
Elevation: 29m					Location: Sta. 10+38m, Offset 9.5m R of CL	Test Hole H-11-97
Start: 11/21/97					Finish: 11/21/97	Water Level: Not Measured
						Sheet 2 of 2
	36.5				wet, medium dense, with silt (FILL)	
40.0	40.0 - 41.5	S-8	0.6	8-10-11	SAND, (SP), very fine, dark gray, wet, medium dense, with trace of silt and some gravel	
					END OF TEST HOLE AT 41.5 FEET	Stopped drilling at 4:00 pm on 11/21/97
45.0						
50.0						
55.0						
60.0						
65.0						
70.0						

NOTES:

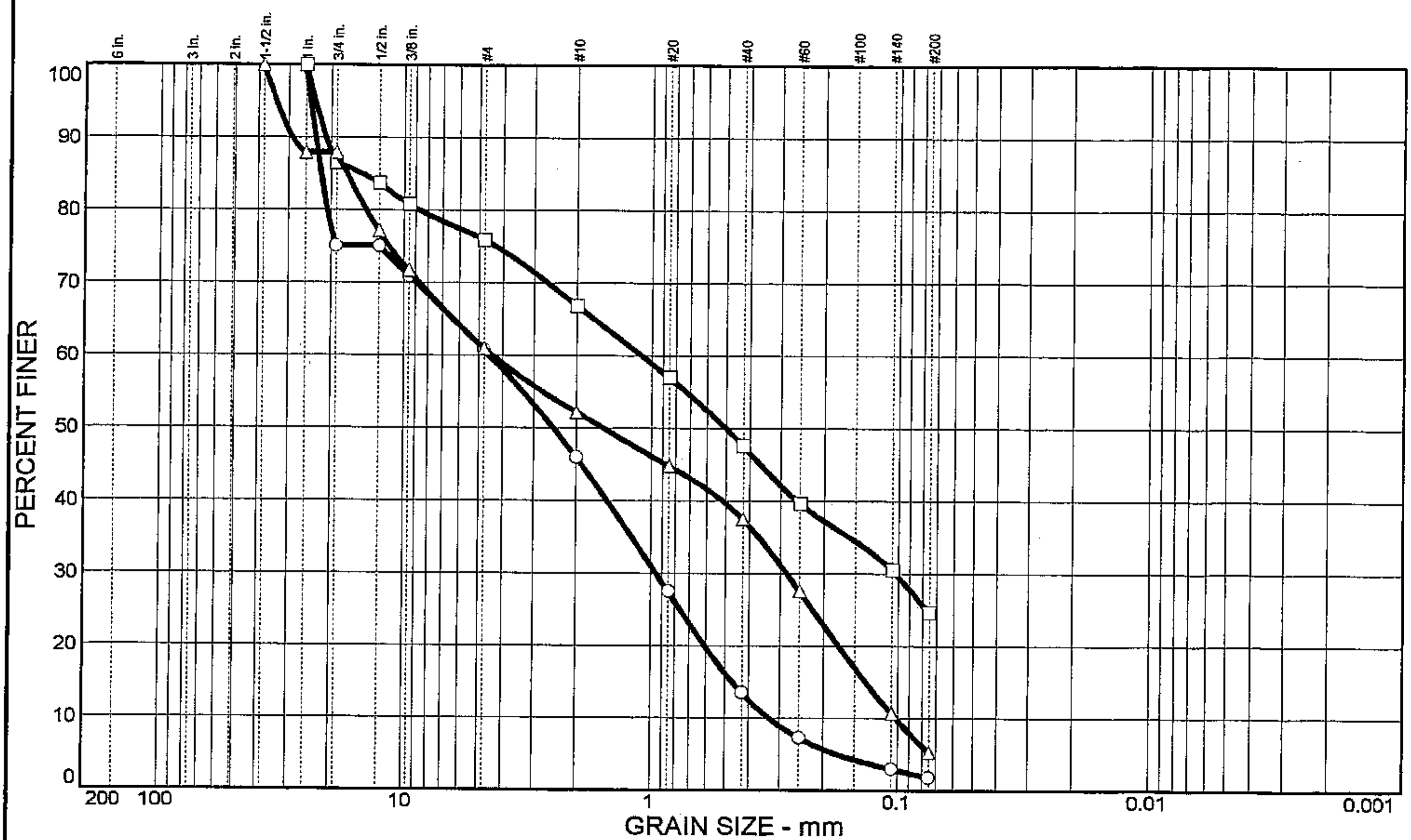
1 foot = 0.3048 meters
1 inch = 25.4 millimeters

1997 LABORATORY TEST DATA

FOR

W-N RAMP

PARTICLE SIZE DISTRIBUTION TEST REPORT



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○		39.2	59.0			SP			
□		24.1	51.3			SM			
△		39.0	55.7			SP-SM			

SIEVE inches size	PERCENT FINER ○	□	△
1.5			100.0
1	100.0	100.0	87.9
.75	75.1	86.5	87.9
.5	75.1	83.7	77.2
.375	71.0	80.8	71.7
GRAIN SIZE			
D_{60}	4.51	1.09	4.38
D_{30}	0.950	0.103	0.281
D_{10}	0.329		0.102
COEFFICIENTS			
C_c	0.61		0.18
C_u	13.70		43.14

SIEVE number size	PERCENT FINER ○	□	△
#4	60.8	75.9	61.0
#10	46.0	66.9	52.2
#20	27.5	57.0	44.8
#40	13.5	47.6	37.5
#60	7.3	39.7	27.6
#140	3.0	30.5	10.7
#200	1.8	24.6	5.3

SOIL DESCRIPTION

○ Poorly graded sand with gravel

□ Silty sand with gravel

△ Poorly graded sand with silt and gravel

REMARKS:

○

□

△

○ Source: H-9 Sample No.: SP-11 Elev./Depth:

□ Source: H-9 Sample No.: SPT-18 Elev./Depth:

△ Source: H-10 Sample No.: SPT-10 Elev./Depth:

SOIL TECHNOLOGY, INC.

Client: CH2MHILL

Project: SR-167, OL-2305
W-N Ramp

Project No.: J-1121

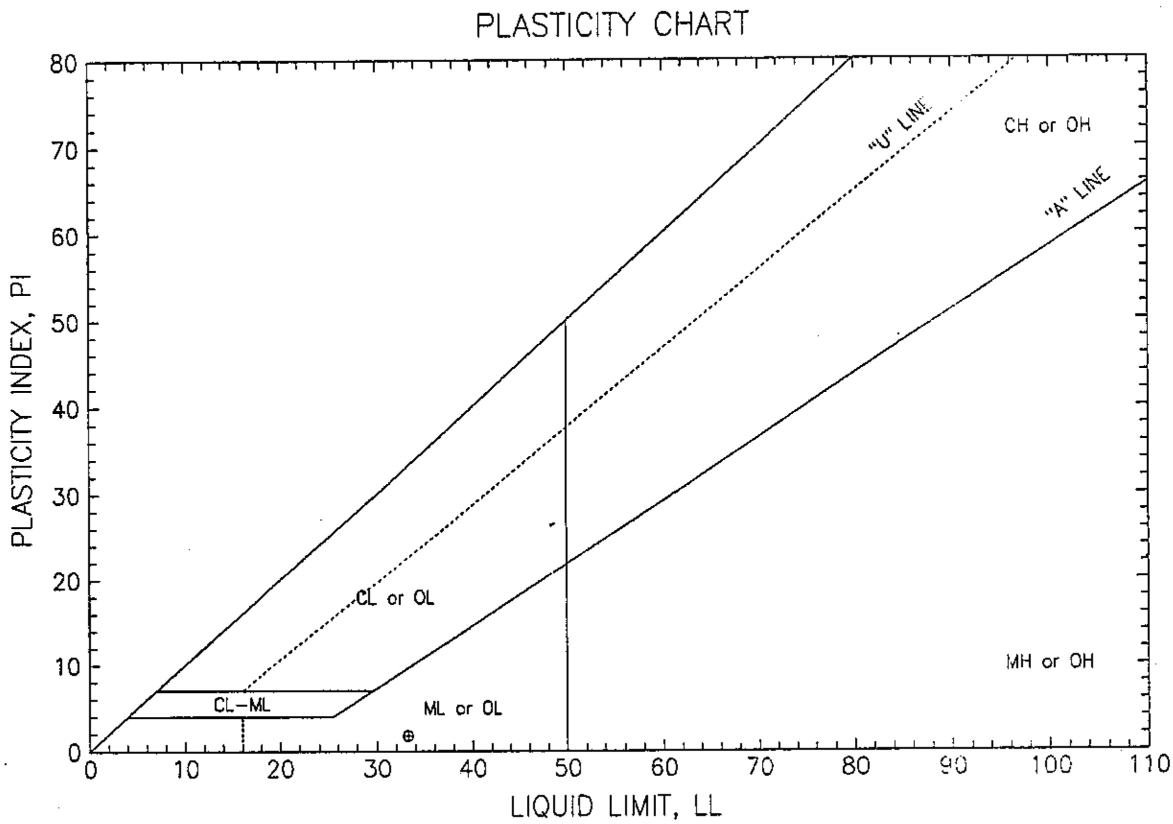
Plate 3

CH2MHILL
SR-167, OL-2305
W-N RAMP

Table 1: % Finer than .75 micron

Soil Boring No.	Sample No.	% Finer than .75 micron
H-8	SPT-2	34
H-9	SPT-4	29
H-10	SPT-3	58
H-10	SPT-11	48
H-11	SPT-5	19

Soil Technology, Inc.	Project : W-N Ramp Project No. : SR-167,OL-2305 Location : Auburn, WA Date : Mon Jan 12 1998



Symbol	Sample Number	Water Content	Liquid Limit	Plastic Limit	Plasticity Index	Soil Classification
O	H9,SPT15	38	33	31	2	(ML) Sandy silt

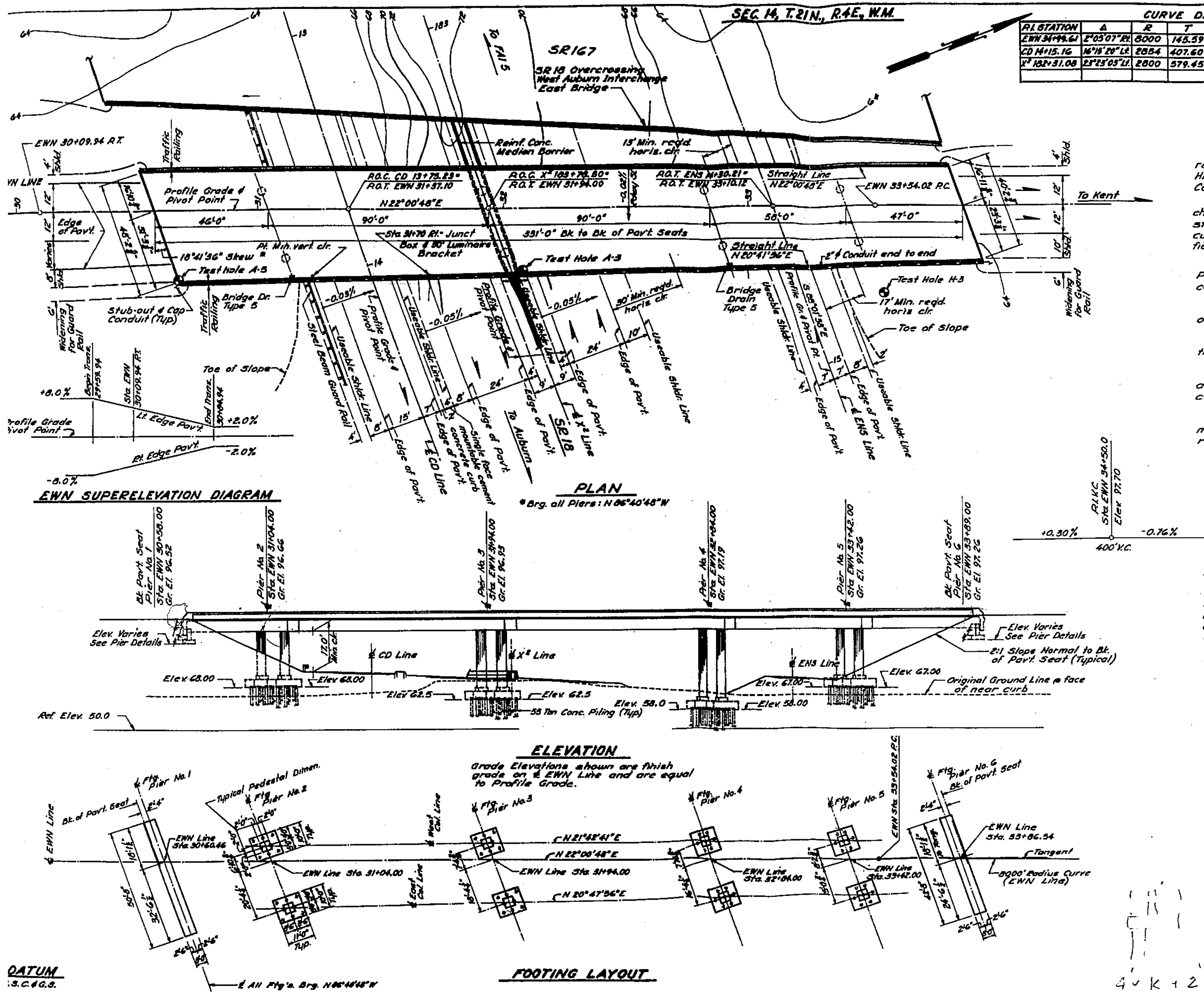
Figure 2

EXISTING DATA

FOR

W-N RAMP

FED ROAD DIV NO	STATE	FED AID PROJ NO	SHEET NO	TOTAL SHEETS
R	WASH.	U-021-1(3)	149	170



CURVE DATA

P.I. STATION	Δ	R	T	L	S	Bk. TAN. BRG.
EWN 34+99.61	2°05'07" Rt.	8000	145.59	291.14	0.02%	N22°00'48"E
CD 14+15.16	16°15'20" Lt.	2854	407.60	809.72	0.03%	S78°49'08"E
X² 182+31.08	23°23'05" Lt.	2800	579.45	1142.76	0.05%	S71°41'25"E

GENERAL NOTES

All material and work shall be in accordance with the requirements of the State of Washington, Department of Highways, Standard Specifications for Road and Bridge Construction, dated 1969.

Footing elevations and substructure details are subject to change depending upon foundation material encountered. Reinforcing steel for the footings, columns and end pier walls shall not be cut until final footing elevations have been determined in the field, and substructure details have been modified as required.

The concrete in the footings of all piers and the walls of Piers No. 1 and 6 shall be Class B mix. All other cast in place concrete shall be Class AX mix.

Falsework shall be carefully released to prevent impact or undue stresses in the structure.

The maximum design soil pressure per square foot is three (3) tons for Piers No. 1 and 6.

Each pile for Piers No. 2, 3, 4 and 5 shall be driven to a depth sufficient to develope a minimum load bearing capacity of fifty-five (55) tons.

Unless otherwise shown on the plans, concrete cover measured from the face of concrete to the face of any reinforcement bar shall be 1½".

APPROXIMATE QUANTITIES

Structure Excavation Class A		680	Cu Yds
Furnishing and Driving Concrete Test Piles 55 Ton		4	Only
Furnishing Concrete Piling 55 Ton		[illegible],900	Lin. Ft
Driving Concrete Piles 55 Ton		60	Only
Steel Reinforcing Bars		83,000	Lbs
Concrete Class B		175	Cu Yds.
Concrete Class AX		50	Cu Yds.
Superstructure - EWN Ramp over SR18			Lump Sum
Downspouts		65	Lin Ft.
Water Reducing Additive	Est.	665	Dollars
Concrete Median Barrier			Lump Sum

LOADING: HS-20
OR
TWO 24 KIP AXLES @ 4' CTRS.

SR 167 **MP 13.77 TO MP 14.73**

15TH ST. S.W. TO W. MAIN ST. IN AUBURN

KING COUNTY

EWN RAMP OVER SR 18

LAYOUT

WASHINGTON STATE HIGHWAY COMMISSION
DEPARTMENT OF HIGHWAYS
OLYMPIA, WASHINGTON

CONTRACT NUMBER 9236

APPROVED October 4, 1971

SHEET 149 OF 170 SHEETS

4 + K + 2 + K + 2

4 + 4 3 K

SR 167/51

81/74

W-N Ramp

HY Form 351-003 (H. F. 26.66)
(Revised 5-67).

WASHINGTON
STATE HIGHWAY COMMISSION
DEPARTMENT OF HIGHWAYS

LOG OF TEST BORING

S.H. ______ S.R. 167 Section West Auburn Interch. Job No. L-3598
Hole No. A-5 Sub Section EWN Ramp Over SR 18 Cont. Sec. 176503
Station EWN 30+67 Pier #1 Offset 29' Rt. ℄ Ground El. 64
Type of Boring Wash Bore Casing 3" X 100' W.T. El. 64
Inspector ______ Date 29 April-May 5, 1971 Sheet 1 of 5

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
			A B U-1	SILT - Brown, organic
				PEAT - Dark brown
			1 Std. Pen	
	1/2'		1 2	SILT - Gray
5			A B C U-3	SAND - Gray
			D	
			4 Std. 7 Pen	
	13		6 6 4	
10				SILT - Gray, layered with extremely fine sandy silt
			A B C U-5	Sand layers, scattered organic matter
			D	
			4 Std. 6 Pen	
	12		6 10 6	SAND - Gray
15				
			U-7	
			9 Std. 10 Pen	SILT - extremely fine sandy & extremely fine
	23		13 13 8	Sand, gray, layered, scattered organic
20				matter

SR 167 MP 13.77 to MP 14.73
15th St. S. W. to W. Main St. in Auburn
-021-1(3) - 1971

Form 351-003-a (H. F. 26.56-A)
Revised 5-67.

Hole No. A-5 Sub Section EWN Ramp Over SR 18 Sheet 2 of 5

Depth	Blows per ft.	Profile	Sample tube nos.	Description of material
			A B C U-9	
			3 2 Std. Pen	SAND - fine to medium, scattered coarse
	5		3 3 10	Clean dark gray, scattered wood
				fragments &/or logs
25			C D E U-11	
	6		3 3 3 2 Std. Pen 12	
30			U-13	35' noted slight artesian, with 1' head
				nearly stopped
	22		8 9 13 22 Std. Pen 14	Gravel, sandy, gray
				appears water bearing
35	23		25 12 11 11 Std. Pen 15	
40	25		24 13 12 12 Std. Pen 16	Sand - fine to coarse & pea gravel, gray
45				

Y Form 351 293-o (H. F. 25.66-A).
Revised 5-67.

Hole No. A-5 Sub Section EWN Ramp Over SR 18 Sheet 3 of 5

	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
			B C U-17	
			D E	Sandy gravel - gray, scattered wood [illegible] lenses
			6 Std.	
			6 Pen	
	12		6	
			1 18	
50				Sand - gray, trace wood, gravel & silt
			U-19	lenses
			5 Std.	
			3 Pen	
	5		2	
			3 20	
55				Gravel - all sizes, sandy, slightly silty
			U-21	
			13 Std.	
			11 Pen	
	21		10	
			11 22	
60				
			U-23	
			10 Std.	
			12 Pen	
	25		13	
			13 24	
65				
			6 Std.	
	11		5 Pen	
			1	
	1		25	Gravelly, silty sand - with layers of
				fine to coarse sand & pea gravel gray
70				

WY Form 351 903-a (H. F. 26.06-A).
Revised 5-67.

Hole No. A-5 Sub Section EWN Ramp over SR 18 Sheet 4 of 5

	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
			U-26	Gravelly, silty sand - with layers of fine to coarse sand & pea gravel, gray
	22		11, 13, 9, 7 Std. Pen 27	
75	21		16, 10, 11, 14 Std. Pen 28	
80	16		11, 8, 8, 9 Std. Pen 29	
85	12		10, 6, 6, 9 Std. Pen 30	
90	25		19, 19, 6, 6 Std. Pen 31	

-WY Form 351-003-o (H. F. 26.66-A).
Revised 5-67.

Hole No. A-5 Sub Section EWN Ramp Over SR 18 Sheet 5 of 5

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	13		18 7 6 6 Std. Pen 32	
				Gravelly silty sand - fine to coarse sand
100				Gray, silty, gravels scattered throughout
	43		42 11 32 18 Std. Pen 33	trace wood
105				
	20		8 10 10 13 Std. Pen 34	
				Sandy gravel, silty binder, greenish
				gray, very dense
110				
	83		52 36 47 34 Std. Pen 35	
	81		42 51 30 75 Std. Pen 36	STOPPED TEST BORING AT 114'-6"
				Logs &/or wood fragments possible throughout
				all samples damp to wet
				no water lost in hole 0' to 114'-6"

167 MP 13.77 to MP 14.73
5th St. S.W. to W. Main St. in Auburn
-021-1(3) - 1971

Log of Test Boring
72 of 88

W-N-Ramp

WY Form 351-003 (H. F. 26.66)
(Revised 5-67).

WASHINGTON
STATE HIGHWAY COMMISSION
DEPARTMENT OF HIGHWAYS

LOG OF TEST BORING

S.H. ______ S.R. 167 Section West Auburn Interchange Job No. L-3598
Hole No. A-3 Sub Section EWN Ramp Over SR 18; Pier 3 Cont. Sec. 176503-U
Station 32+07 Offset 26' Rt. ℄ Ground El. 68.4
Type of Boring Auger Casing 122'-0" (auger) W.T. El. 61.4
Inspector ______ Date 19 April, 1971 Sheet 1 of 6

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
				3" asphalt surfacing FILL - Sand & gravel, all sizes with sparse cobbles,
				silty, brown & gray, compact to dense, moist to
			13 Std.	
			18 Pen	2' wet thereon
	36		18 1	
			33	logs or wood fragments possible through the depth
5				of this boring
			9 Std.	
	20		11 Pen	
			5 2	
	11		6	SAND - fine, silty with trace of peat, grey, loose
10				
			C U-3	
			D	
			E	SAND - fine to coarse, silty with occasional silt lense
			8 Std.	
15	25		12 Pen	trace of peat & wood fragments, grey, compact
			13 4	
			18	
			B	
			C U-5	SILT & SILTY FINE SAND - marbled & layered
			D	
			E	trace of peat, grey, slightly compact
			9 Std.	
20	19		11 Pen 6	

.67 MP 13.77 to MP 14.73
5th St. S. W. to W. Main St. in Auburn
-021-1(3) - 1971

Hole No. A-3 Sub Section EWN Ramp Over SR 18 Sheet 2 of 6

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.		DESCRIPTION OF MATERIAL
			8		
			17		
			B		
				U-7	SAND - fine, silty thin lenses of silt & trace of peat
			8	Std.	
	35		15	Pen	grey, slightly compact to compact
			20	8	
25			27		
			6	Std.	
	19		9	Pen	
			10	9	
			12		
30					
			12	Std.	
	23		11	Pen	
			6	10	
	12		6		SILT & GRAVELLY SAND - Layered, fine to coarse
35					silty sand with small amount of fine gravel, grey
					slightly compact
			16	Std.	
	30		14	Pen	GRAVELLY SAND - all sizes sand with fine gravel &
			16	11	
			24		sparse course gravel, slightly silty, grey, compact to dense
40					
			27	Std.	
	58		28	Pen	
			30	12	
			38		
45					

NY Form 351-003-a (H. F. 26.6a-r.)
Revised 5-67.

Hole No. A-3 Sub Section EWN Ramp over SR 18

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	26		8 13 13 13 Std. Pen 13	
50				
	44		14 20 24 23 Std. Pen 14	SAND - fine, silty with silt stringers, grey, compact to dense
55				
	30		14 16 10 10 Std. Pen 15	
	20			SAND & SILT - Layered with trace of peat, sand is fine & silty, grey, slightly compact to compact
60				
	20		6 8 12 16 Std. Pen 16	
65				
	20		6 9 11 16 Std. Pen 17	
.0				

R 167 MP 13.77 to MP 14.73
5th St. S.W. to W. Main St. in Auburn
-021-1(3) - 1971

Log of Test Borings

Form 351-003-a (H. F. 26.66-A).
Revised 5-67.

Hole No. A-3 Sub Section EWN Ramp over SR 18 Sheet 4 of 6

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	16		6 8 8 10 Std. Pen 18	GRAVELLY, SILTY SAND - All sizes, trace of peat & wood fragments, grey, slightly compact to compact
75				
	16		8 11 5 5 Std. Pen 19	
80				
	10		7 5 5 8 Std. Pen 20	
85				
	15		4 7 8 11 Std. Pen 21	
90				
	28		11 10 18 24 Std. Pen 22	
5				

VY Form 351-003-a (H. F. 26.66-A).
Revised 5-67.

Hole No. A-3 Sub Section EWN Ramp over SR 18 Sheet 5 of 6

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	16		6, 6, 10, 13 Std. Pen 23	
100				
	14		8, 7, 7, 7 Std. Pen 24	
105				
	15		5, 7, 8, 8 Std. Pen 25	
110				
	28		12, 16 Std. Pen 26	
	86/9"		32, 54/3"	SANDY GRAVEL - Silty, all sizes with cobbles grey, dense to very dense
115				
	124		70, 54 Std. Pen 27	
20				

HWY Form 351-003-a (H. F. 26.6a-A1.
Revised 5-67.

Hole No. A-3 Sub Section EWN Ramp over SR 18 Sheet 6 of 6

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
			17 Std.	
	78		28 Pen	
			50 28	TEST BORING STOPPED AT 123'-6"
125				
130				

SR 167 MP 13.77 to MP 14.73
15th St. S. W. to W. Main St. in Auburn
-021-1(3) - 1971

Log of Test Borings

W-N-Ramp

Form 351-003 (H. F. 26.66)
(Revised 5-67).

WASHINGTON
STATE HIGHWAY COMMISSION
DEPARTMENT OF HIGHWAYS

LOG OF TEST BORING

S.H. ____ S.R. 167 Section West Auburn Interchange Job No. L-3598

Hole No. H-3 Sub Section ~~Ramp ... Xing~~ EWN/SR 18 Cont. Sec. 1765

Station ~~L-184+73~~ EWN 33+57 Pier #3 Offset 5' Lt. ℄ 34' Lt Ground El. 65

Type of Boring Wash Bore Casing 3" X 45' W.T. El. 64

Inspector ____ Date 20 & 21 Nov. 1969 Sheet 1 of 6

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
			B C U-1	SILT - Organic, brown, wet
			D E	PEAT - Wood, organic matter, brown, wet
	1		1 Std. Pen 2	
			3	
	6		3	SAND - Scattered silt lenses, wood
5				gray, wet
			C U-3	
			D E	
			4 Std.	
	13		7 Pen	
			6	
			6 4	
10				
			B C U-5	
			3 Std.	
	9		4 Pen	
			5	
			3 6	
15				
			A B C U-7	
			5 Std.	
	13		8 Pen	SAND - fine to coarse, scattered wood,
			11	
	25		14 8	gravel, trace silt, gray, wet

167 MP 13.77 to MP 14.73
5th St. S. W. to W. Main St. in Auburn
021-1(3) - 1971

No. H-3 Sub Section Ramp L U'Xing (FUN/SRIE) Sheet 2 of 6

TH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
			U-9	SAND - fine to coarse, scattered wood, Gravel, trace silt, gray wet
	40		11 17 23 23 Std. Pen 10	SAND - fine to medium, scattered coarse, scattered wood, slightly silty, trace of gravel, damp
25	16		6 6 10 17 Std. Pen 11	
				SAND - fine to medium, gray, wet
30	26		7 11 15 19 Std. Pen 12	
35	25		6 11 14 18 Std. Pen 13	
40	19		3 10 9 13 Std. Pen 14	

Form 351-003-o (H. F. 26.66-A).
Revised 5-67.

Hole No. H-3 Sub Section Ramp L Under Crossing (EWN/SKIE) Sheet 3 of 6

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
			11 Std.	
			5 Pen	Water level - -O.D.' (ground level)
	9		4 15	
			6	SILT - compact, gray, moist
				Logs or wood fragments possible through
				the depth of this boring.
50			B C D U-16	
			14 Std.	
			14 Pen	SILTY, GRAVELLY, FINE SAND - compact, damp
	24		10 17	
			12	sparsely scattered fine gravel, a trace of
			20 Std.	peat, gray silt, brown peat
55			13 Pen	
	29		16 18	
			19	
				SILT WITH PEAT LENSES - slightly compact to
				compact, moist, gray silt, 2" ± lenses
			9 Std.	of brown peat and mottled grayish brown
60			14 Pen	silt
	30		16 19	
			33	
				SILT - slightly compact, moist, mottled gray
				green and brown
			6 Std.	
65			9 Pen	SILTY, SANDY GRAVEL - Loose to slightly compact,
	16		7 20	
			11	gray, wet, fine to coarse sand and
				fine to coarse gravel. Fragments of
				wood and traces of peat scattered through
70			3	

WY Form 351-003-a (H. F. 25.66-A).
Revised 5-67.

...e No. H-3 Sub Section Ramp L Under Crossing CLW/SKLS Sheet 4 of 6

...TH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	6		3 3 5 Std. Pen 21	
75				
	7		3 5 2 4 Std. Pen 22	
80			A B C D U-23	
	14		6 7 7 7 Std. Pen 24	
85				
90	13		6 7 6 7 Std. Pen 25	

Y Form 351-C03-a (H. F. 26.66-A).
Revised 5-67.

Hole No. H-3 Sub Section Ramp L Under Crossing (EWN/SR/E) Sheet 5 of 6

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
				At about 97' - penetrometer blow-count ranges from compact to very dense.
100	52		54 25 27 13 Std. Pen 26	
105	45		12 16 29 12 Std. Pen 27	
				SILTY SANDY GRAVEL - Dense, damp, gray fine to coarse sand, fine to coarse gravel and cobbles
110	75		31 15 60 Std. Pen 28	
				(Dynamite used at 112'0")
115	51		35 27 24 20 Std. Pen 29	
120			20	

Hole No. H-3 Sub Section Ramp L Under Crossing (EWN/SR18) Sheet 6 of 6

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.		DESCRIPTION OF MATERIAL
	53		21 32	Std. Pen	
			50/3"	30	TEST BORING STOPPED AT 121'3"
125					

Chapter 7 -- References

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